PREDICTION OF HYDRAULIC CONDUCTIVITY OF CLAY LINERS: A FIELD AND LABORATORY STUDY

By

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To All Al-musawe Family Members:
Now and forever with all my love.

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LIST OF SYMBOLS

A	Cross Sectional Area of Soil Sample
A_c	Percent Activity of Soil
A_d	Discharge Area and Equal to A
A_r	Percent Area Ratio
A_s	Seepage Area
a	Cross Sectional Area of the Small Standpipe
D	Diameter of Soil Sample
e	Void Ratio of Soil
G_s	Specific Gravity of Soil Solid
H_o	Hydraulic Head Difference Applied to Soil Sample
H_s	Suction Head Within Soil Sample
i	Hydraulic Gradient
K	Steady State/Saturated Hydraulic Conductivity (Permeability)
K_i	Transient Hydraulic Conductivity/Coefficient of Infiltration
L	Length of Soil Sample
LL	Percent Liquid Limit
n	Percent Porosity
n_e	Percent Effective Porosity

PL	Percent Plastic Limit
PI	Percent Plasticity Index
Q	Quantity of Water Discharged
R	Drainage Impedance
S	Percent Degree of Saturated
T	Time
V	Velocity of Discharge
V_s	Velocity of Seepage Discharge
w	Percent Moisture Content
γ_d	Dry Unit Weight
γ_w	Wet/Moist/In-situ Unit Weight

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Low hydraulic conductivity clay soils are used in landfills to impede the movement of leachate down to the natural groundwater table. Hence, hydraulic conductivity is an important soil property in the design and assessment of liner thicknesses and integrity. Because of the sensitivity of hydraulic conductivity to many factors, there are no standard laboratory or field testing methods, and therefore, there exists wide variation in predicted values.

A local natural clay soil, "Terra-Seal Natura Premix®," was used for this study because this soil is used in the construction of a number of landfills in Florida. A series of rigid-wall permeameter tests were performed for a quantitative prediction of hydraulic conductivity variation as a function of sample thickness, number of layers,

hydraulic gradient, porosity, degree of saturation, dry unit weight, and time. The variation of moisture content versus depth for the fully saturated samples was found to vary significantly. Measurements were also made of the variation of partially saturated soil suction with dry unit weight and moisture content.

A number of field infiltration tests were conducted at two existing landfill projects, and coefficients of hydraulic infiltration were measured. Using these values and the amount of suction obtained in the laboratory, saturated hydraulic conductivities were predicted. These predictions agreed very closely with those obtained in the laboratory by the author and others.

Desiccation cracks, depth versus dry unit weight, and moisture content variations were studied in the field. Three test strips, constructed using one, two, and three layers, were subjected to equal compactive energies. Variations of dry unit weight and moisture content with depth were found to be the least for the one layer strip. The surfaces of all test strips were cracks, of which the depth and width varied. Covering the test strips with Visqueen did not prevent cracking but did minimize it. In the laboratory, the effect of these cracks on the hydraulic conductivity diminishes after 16 hours of testing. Field hydraulic conductivity can be predicted accurately and efficiently by the method developed in this study.

CHAPTER 1

LITERATURE REVIEW, BASIC CONCEPTS, AND PURPOSE AND SCOPE OF THIS STUDY

Definition of the Problem

Hydraulic conductivity of soil has become the most important property in geotechnical and geo-environmental engineering, agronomy, agriculture, and in all fields that involve seepage and drainage of water and industrial liquids through soil. Yet it is the most varied, least known, least studied, and most difficult soil property to determine. In one of the geotechnical and geo-environmental engineering areas where a reliable and accurate estimate of the hydraulic conductivity is most needed is in the determination of the clay liner thicknesses. Clay liner is a soil layer of certain thickness consisting of sandy silty clay with low hydraulic conductivity. Clay-lined facilities have been used extensively for the containment and disposal of hazardous and nonhazardous solid and liquid waste. Occasionally slowly permeable natural clay-rich deposits were relied upon to retard the movement of leachate and liquids from landfills or surface impoundments. Presently, in most cases, remolded layers of soils with laboratory hydraulic conductivities of

$1 * 10^{-7}$ cm/s or less have been used with the intention of retaining leachate and liquids.

There is an increasing body of data which indicates that hydraulic conductivity of in situ (recompacted) clays may be greater than those measured on samples in the laboratory (Daniel 1987, Mitchell 1976, Schmid 1966, Sowers 1979). Although there is no set standard for laboratory hydraulic conductivity tests on clays, all existing methods yield comparable values. Major errors in the laboratory values are due to the large sample disturbances, relatively small dimensions of the tested samples, and the very large applied hydraulic gradient.

On the other hand, proposed field methods are complicated, difficult to run, time consuming, require lengthy analysis, require highly technical personnel, very sensitive to minor errors in the setup, and do not resemble a laboratory setup (Chen et al. 1986, University of Texas, College of Engineering 1990, Gorden et al. 1989, Hamilton et al. 1979, Mitchell 1976, Olsen and Daniel 1979, Peirce et al. 1987(b), Schmid 1966, Stewart and Nolan 1987, Wit 1966). The major errors in field values of hydraulic conductivity are mainly caused by soil suction (capillary pressure) which is due to incomplete saturation of the soil and the ability of the permeant liquid (water) to travel in both vertical and horizontal directions (Daniel 1984, Stewart and Nolan 1987).

Clay Liner and Landfill Technology

A clay liner (sometimes referred to as soil or earthen liner) may be manmade (compacted), or a naturally occurring deposit (not disturbed). Natural clay liners are formed by aquitards or aquiclude. Wastes may be buried wholly within a natural clay liner (Fig. 1A), partially within a natural liner (Fig. 1B), or as in Fig. 1A and Fig. 1B but not within a natural liner (Fig. 1C).

Manmade liners consist of a horizontal liner, an inclined liner, or a cover over a landfill (Fig. 2). The soil in these liners can be either naturally occurring soils or manmade soils by the mixing of natural soils with one or more different materials. In either case, the soils must meet set specifications concerning fineness content, clay content, plasticity index, liquid limit, and moisture content. The soil then is placed in horizontal layers with suitable thicknesses and compacted to achieve a certain dry unit weight.

A typical section of a landfill containment system, including typical dimensions of various components, is shown in Fig. 3. The clay liner impedes or controls outward seepage of contaminant-laden fluids from the structure. The leachate collection and removal system conveys fluids off the clay liner to collection sumps and where the liquid is removed. The final cover impedes or eliminates infiltration of meteoric water into the refuse, thereby controlling

leachate generation. The entire concept of waste containment is basically the successful interruption of the natural hydrological cycle, as depicted in Fig. 4.

Background Information of Previous Work Related to this Study

This investigation deals with the hydraulic conductivity of naturally occurring soils as clay liners. Therefore, only similar previous work will be dealt with in this literature review. Note, however, that the concepts are the same in either case. Although the principle of hydraulic conductivity was recognized in 1911, its application to landfill liners became extensive in the last 10 years when landfill technology started to surface. Early work included studies of simple prediction of the transient time of a wetting front and the seepage rate after achieving saturation (Green and Ampt 1911). This study is still frequently used and commonly referred to as the Green-Ampt model.

Hydraulic Conductivity of Saturated Clay Soils

Hydraulic conductivity is the speed with which water flows through soil media under unit hydraulic gradient. The laws by which this flow takes place are very well understood in sand and coarser grained soils but are still under debate for clay soils. Flow can be classified as one-, two-, or three-dimensional. One-dimensional flow is flow in which all the fluid parameters, such as pressure, velocity,

temperature, etc., are constant in any cross section perpendicular to the direction of flow. These parameters can vary from section to section along the direction of flow but are generally assumed to be constant. This in turn means that the soil media is assumed to be homogeneous. In two-dimensional flow, the fluid parameters are the same in parallel planes, whereas in three-dimensional flow, the fluid parameters vary in three coordinate directions. For the purpose of analysis, in all the literature reviewed and in all geotechnical engineering applications, flow problems are assumed to be at most two-dimensional.

Flow can also be described as laminar (zone I, Fig. 5), where the fluid flows in parallel layers without mixing, or turbulent (zone III, Fig. 5), where random velocity fluctuations result in mixing of fluid and internal energy dissipation. There can also be intermediate or transition states between laminar and turbulent flow. These states are shown in Fig. 5. The flow in most soils is considered laminar when the particle size is less than 0.05 cm and uniform size, with a low seepage velocity, and a hydraulic gradient (i) of one (Holtz and Kovacs 1981, Mitchell 1976, Lambe and Whitman 1979, Sing 1967, Taylor 1948). In case of clays, the flow is laminar when the particle size is 0.0002 cm or less, particles are not of uniform dimension, the hydraulic gradient is always much greater than one, and the

seepage velocity is very high. D'Arcy (1856) showed experimentally that for clean sands in zone I,

$$V = K * i \quad (1)$$

(Darcy's Law) where

K = hydraulic conductivity, saturated hydraulic conductivity, Darcy coefficient of permeability, or permeability (cm/s),

V = Q/A*T = discharge velocity (cm/s),

i = H_o/L = hydraulic gradient (cm/cm).

Therefore, equation 1 becomes

$$K = (Q * L)/(A * T * H_o) \quad (2)$$

where

Q = quantity of discharge (cm^3),

A = cross sectional area of soil (cm^2),

T = time (s),

H_o = hydraulic head difference applied to soil (cm), and

L = length of flow path in soil (cm).

Another concept in fluid mechanics is the law of conservation of mass, and for incompressible steady state flow; this law reduces to the equation of continuity:

$$q = Q/T = V_1 * A_1 = V_2 * A_2 = \text{constant} \quad (3)$$

where

q = rate of discharge (cm^3/s),

V_1, V_2 = velocities of flow at section 1 and 2,

A_1, A_2 = cross sectional areas of soil at section 1 and 2 .

The velocities of flow outside the soil media (V or V_d) are not the same as the seepage velocity (V_s) of flow inside the soil media, or

$$V = V_d = n * V_s, \quad (4)$$

where n = percent porosity, and since the water can only seep through the connected pores,

$$V = V_d = n_e * V_s, \quad (5)$$

where n_e = percent effective porosity

Using the above notations and combining equations 1, 2, and 3,

$$q = V * A = \{ [Q / (T * A * n_e)]\} *$$
$$* A = (K * H_o * A)/L, \quad (6)$$

and by simplifying,

$$K = (Q * L)/(A * T * H_o * n_e) \quad (7)$$

Currently only equations 1 and 2 are used to obtain hydraulic conductivity for all types of soil, including clays.

Prediction of Hydraulic Conductivity of Saturated Clays

Techniques for measuring hydraulic conductivity in coarse-grained soils (falling head or constant head tests in the laboratory, and pump tests from wells in the field) are very well established and standardized. Techniques for measuring hydraulic conductivity in fine-grained (clays) soils, however, are not very well known or standardized. The reason for this is that past practice frequently has been to assume that clays are effectively "impervious," and therefore, attempts to measure their hydraulic conductivity were not undertaken. But with the progression of landfill use and technology, the need for measuring hydraulic conductivity has become very important and vital in order to monitor the water and leachate movements over many hundreds of years, thereby providing parameters necessary to protect the integrity of the groundwater below the landfill.

Empirical methods

Many empirical formulas exist for predicting the saturated hydraulic conductivity of soils with a particle size greater than 0.0002 cm (sand). This is mainly because the sand has particles that are uniformly distributed and spherical in shape which result in pores that are relatively uniform in size and distribution. Clays, on the other hand, have particles that are flaky, less than 0.0002 cm in size, have large electrically charged surface areas, and many types of pores with different sizes and distributions. However, there are two known empirical equations that can be used with large inaccuracy to predict the hydraulic conductivity (k) in clayey soils. These are

1. Kozeny-Carman equation (Mitchell 1976, Sing 1967, Taylor 1948):

$$k = K * (\mu/\gamma_w) = [1/(P_o * t^2 * S_o^2)] * [n^3/(1-n)^2] \quad (8)$$

where

k = intrinsic permeability or permeability (terminology used in hydraulic and fluid mechanics engineering) in cm^2,

μ = viscosity of water in g s/cm^2,

γ_w = unit weight of water in g/cm^3,

P_o = pore shape factor (2.5 for sand), and

t = tortuosity factor ($2^{0.5}$ for sand).

2. Loudon's Formula (Sing 1967):

$$\text{Log}_{10} (K * S_s^2) = a + b * n \quad (9)$$

where

S_s = specific surface of soil particle in cm^2/cm^3,

a = constant = 1.365 at 10°C for sand, and

b = constant = 5.15 at 10°C for sand.

Laboratory methods

One-dimensional consolidation cell permeameter. A typical one-dimensional consolidation cell permeameter consists of a 4 to 10 cm diameter by 1.9 to 10 cm high consolidation ring mounted in a cell as shown in Fig. 6. A reservoir of water surrounding the consolidation ring maintains atmospheric pressure at the effluent end (top) of the specimen. The hydraulic pressure at the base of the sample is controlled using the system described by Olsen and Daniel (1979). The hydraulic conductivity can be calculated by

$$K = (C * cc * \gamma_w)/(1 + e) \quad (10)$$

where

C = coefficients of consolidation in cm^2/s,

cc = compressibility in $1/(g/cm^2)$, and

e = void ratio.

Flexible-wall permeameter. Hydraulic conductivity tests with flexible-wall (triaxial type) devices are performed typically using the cell shown in Fig. 7. Interchangeable base pedestals permit testing of specimens with a diameter between 2 and 15.2 cm and with a wide range of heights ranging from 2 cm to 20 cm. Double drainage lines to both the top and bottom of the test specimen are used to flush air bubbles out of the lines. The spare drainage lines are also used in conjunction with an electrical pressure transducer to measure pore-pressure response during back pressuring of the soil, and to measure the pressure drop across the specimen. There are two standard test types that use the flexible-wall permeameter.

1. Constant head: In this test, the hydraulic gradient ($i = H_o/L$) is maintain constant and the volume of discharge (Q) is measured during a time (T). The hydraulic conductivity is calculated using equation 2. For fine-grained soils the constant head is typically applied using a Mariotte bottle similar to that shown in Fig. 9. Such equipment is designed to apply only small heads (a few feet of water) so it is most useful with rather pervious soils or

in a case where prolonged testing times can be tolerated. The main advantages of constant head tests are the simplicity of interpretation of data and the fact that use of constant head minimizes confusion due to changing volume of air bubbles when the soil is not saturated.

2. Falling head: This is a more common test for fine-grained soils in which the time (T) for the hydraulic head to drop from one level (H_{o1}) to a lower level (H_{o2}) in a volumetric tube (typically a pipet or a buret with cross sectional area (a)) due to flow through a soil sample of cross sectional area (A), and length (L), is measured. The hydraulic conductivity is calculated using

$$K = [(a * L)/(A * T)] * \ln H_{o1}/H_{o2} \qquad (11)$$

The advantages of using this procedure are that small flows are easily measured using the pipet or buret. The observation time may still be long, in which case corrections for water losses due to evaporation or leakage may be added. The testing time may be reduced by increasing the flow rate by superimposing an air pressure (A_p) on top of the water in the pipet, thus increasing the heads by a certain amount equal to A_p/γ_w.

Rigid-wall permeameter. The Rigid-wall (compaction-wall) permeameter consists of a 10 cm diameter compaction mold with variable heights. The mold is clamped between two

acrylic end plates and sealed with either gaskets or O-rings, as indicated in Fig. 8. The soil is either allowed or not allowed to swell. The influent water is usually stored in a separate device which contains an air-water interface inside a glass pipet. The pressure acting on the water is controlled with an air pressure regulator. Flow quantities are measured by reading the position of the air-water interface inside the pipet. The effluent water is collected in a reservoir that is open to atmospheric pressure. The drainage line leading to the permeameter is saturated with water, but no back pressure is applied nor is the effluent line de-aired. Both falling and constant head test methods can be used with the rigid-wall permeameter as described above together. The same equations are used to calculate the hydraulic conductivity (i.e., equation 11).

Field methods

The soils in landfill liners are not saturated soils and, therefore, cannot be considered saturated soils. Consequently, the existing field methods of determining hydraulic conductivity for saturated soils cannot be used. However, if the soils are saturated, then the hydraulic conductivity can be measured in the field by drilling a hole in the ground, measuring the rate of flow of water into or out of the hole and using an appropriate formula to calculate the conductivity (Harr 1962, Lambe and Whitman 1979, Olsen

and Daniel 1979, Schmid 1966). Tests may be performed at a constant head by establishing a high head of water in the borehole and pumping at a rate sufficient to maintain this head. Also, tests may be made with a variable head, that is, with the head set at a nonequilibrium value initially and then measured as a function of time with no further pumping.

Other field test methods are used and sometimes erroneously called "field hydraulic conductivity tests." These are actually field infiltration tests (ASTM 1989, Bagchi et al. 1985, Bond and Collis-George 1981, University of Texas, College of Engineering, 1990, Daniel 1984, Gorden et al. 1989, Hamilton et al. 1979, Kraatz 1977, Stewart and Nolan 1987). This is because the soil below the testing apparatus cannot be completely saturated. Two relatively simple test set-ups, single and double ring infiltrometer, are shown in Fig. 10. A more complex setup is shown in Fig. 11 (University of Florida, Department of Geology and Civil Engineering, 1990). In all three test arrangements the function of the outer ring is to prevent lateral flow of water during the tests. All suggested tests are complicated, very sensitive, time consuming, not rugged, expensive, require lengthy analysis, and require a highly technical person to perform them.

Prediction of Hydraulic Conductivity in Partially Saturated Clays

A practical science for prediction of moisture migration in partially saturated soils has not been fully developed for unsaturated soils for two main reasons. First, there has been a lack of an appropriate science with a theoretical base. Second, there has been a lack of practical technology to render engineering practice economically viable. There is a need for further experimental studies and case histories to substantiate the available concepts and theories (Fredlund 1979). This summary includes a brief review of the concepts of moisture flow in partially saturated soils, including analysis techniques for application to geotechnical problems.

Basic concepts and definitions

Water in soil is continuously under the influence of one or more forces that determine its energy status or potential. There are four types of potential gradients that cause flow of water through soil--hydraulic, electric, chemical, and thermal. However, under most circumstances the hydraulic and chemical gradients do exist. Hydraulic potential includes the gravitational and matrix components. Chemical potential is often referred to as osmotic potential. The total potential is the sum of the component potentials, or

$$\phi_{total} = \phi_h + \phi_e + \phi_c + \phi_t \qquad (12)$$

where

ϕ stands for potential energy.

ϕ_h = hydraulic potential

ϕ_e = electric potential

ϕ_c = chemical potential

ϕ_t = thermal potential

The potential is expressible physically in at least three ways (Cedergren 1977, Harr 1962, Mitchell 1976):

1. Energy per unit mass. This is a fundamental expression of potential, using units of ergs per gram or joules per kilogram.

2. Energy per unit volume. This yields the dimensions of a pressure (e.g., kilopascals, atmospheres, or pounds per square inch). This expression is convenient for the osmotic and pressure potentials.

3. Energy per unit weight (hydraulic head). This is the height of a liquid column corresponding to the given potential. This expression of potential is certainly simpler, and often more convenient, than the preceding expressions. Hence, it is common to characterize the state of soil water potential in terms of water head in centimeters, meters, or feet.

Consideration of the potential is important because of its relation to the movement of water in soils. The gravitational component of potential is due to the continuous

downward action of the Earth's gravitational field. The higher the elevation, the greater the potential. Matric potential is due to capillary action, which in turn depends on the adhesion between soil and water and cohesion between water molecules. If free water is adsorbed by soil without a change in elevation, its potential energy is decreased, the extent of decrease being a function of how tightly the water is attracted to the soil. Matric heads are also referred to as suction heads and are always negative in sign. Matric potential varies directly with the soil water content; that is, as the water content is increased toward saturation, the matric potential increases toward its maximum value, which is zero at full saturation, as shown in Fig. 12 (Hillel 1971).

Osmotic forces represent the attraction between dissolved ions and water. The higher the concentration of ions, the greater the osmotic forces. Like matric forces, osmotic forces reduce the potential of water, which causes the osmotic potential to be negative in sign.

The rate of water flow through soils depends on two factors: (1) the driving force (potential gradient), which is normally taken as the change in water potential per unit of distance, and (2) the conductivity, or the ability of the soil to transmit water. The conductivity, as used here, is analogous to the hydraulic conductivity for saturated flow. The coefficient is multiplied by the gradient to obtain fluid velocity. The higher the water content, the higher the

conductivity. The water content affects the ability of the soil to transmit water for several reasons, principally by influencing the total cross sectional area of pores through which water flows, the amount of friction encountered, which is maximum where water moves in thin films close to soil particle surfaces, and the length of flow path through the pore. Figure 13 (Hillel 1971) shows the typical relationships between conductivity and suction for two partially saturated soils, sand and clay.

Infiltration is the entry of surface applied water into, and its movement through, soil. Infiltration is normally assumed to occur in response to the combined influence of matric and gravitational forces. The advance of water is along a boundary known as the wetting front. During wetting, at least a thin saturated zone is maintained at the surface where water first enters the soil. Since the pores in this zone are water filled, they exhibit a maximum and constant conductivity equal to the hydraulic conductivity. The magnitude of saturated conductivity is very important in determining how fast water can infiltrate and move through the soil. Evidence of this fact is that sands, which have a high hydraulic conductivity when saturated, have a relatively high infiltration rate during wetting.

An important characteristic of soil wetting is that it slows with time. There have been several reasons cited for this decrease in velocity of the wetted front. Colloids in

the soil may swell and reduce the pore size, or fine material from the surface may be washed into the soil, plugging up the pores. The continuous sheet of water above the soil and in the upper layer of soil makes it difficult for the air in the soil to escape and to make room for further water to enter. Potential gradient across the wetted front zone decreases as the potential difference is dissipated over a widening wetted front region (Hillel 1971).

Prediction of moisture flow in partially saturated soils

The success of a field hydraulic conductivity prediction depends quite heavily on the prediction of the depth and extent of the wetted zone, because water is the main factor in saturating the soil, thus allowing the saturated hydraulic conductivity to be measured. The prediction of moisture movement in partially saturated soils is very complicated because of the following potential variabilities associated with the soil, water, and driving forces.

1. Soil type, gradation, structure, and dry unit weight.
2. Amount and type of dissolved salts.
3. Temperature changes in space and time.
4. Moisture changes in space and time.
5. Soil suction and conductivity changes with moisture content, temperature and dissolved salts.
6. Nonlinearity of the conductivity versus soil suction curve.

7. Hysteretic nature of the conductivity versus soil suction relationship.

8. Difficulty in obtaining accurate measurements of soil suction and conductivity.

9. Volume change upon inundation.

10. Sources of moisture differ in their character by way of amount of available water, rate of supply, and location within the soil profile.

11. Soil anisotropy and inhomogeneity.

12. Thickness of soil profile.

13. Water properties change according to temperature, dissolved salts, and capillary attraction.

14. Soil fluids including adsorbed water, free liquid water, water vapor, and air.

15. In situ stress conditions and mechanisms are not easily defined.

16. Boundary conditions for analysis are related to environmental conditions which are difficult to predict.

Since the beginning of the twentieth century, the problem of partially saturated flow has been studied by physicists, soil scientists, hydrologists, petroleum engineers, and geotechnical engineers. The following is a brief review of some of the more known studies.

Buckingham (1907) developed the following equation as the general fluid flow law.

$$Q = \lambda * S \tag{13}$$

where

Q = the mass of water per square centimeter,

S = i = $\partial\psi/\partial x$ = gradient of capillary attraction, and

$\lambda = k_i$ = capillary conductivity = infiltration coefficient.

He noticed that both the capillary conductivity and the soil suction pressure change with water content. Green and Ampt (1911) studied the motion of a wetting front through the soil and developed the following equation:

$$dV/dt = A * (dl/dt) * n \tag{14}$$

where

V = volume of liquid water,

l = depth of water infiltration, and

n = porosity.

The combination of this equation with Poiseuille's law of flow in capillary tubes was used to develop the Green-Ampt wetting front motion equation.

Richards (1931) used the general equation of motion of viscous fluid, the Navier Stokes equation:

$$dv/dt = F \cdot \nabla (P/P_w) + (\mu/P_w) (\nabla \nabla \cdot v/3 + \nabla \cdot \nabla v) \quad (15)$$

where

dv/dt = acceleration,

F = external or body forces = $- \nabla \phi$,

$\nabla P/P_w$ = force due to pressure gradient, and

$(\mu/P_w) (\nabla \nabla \cdot v/3 + \nabla \cdot \nabla v)$ = expression of viscous retarding forces.

$$\nabla = \text{del operator} = \partial/\partial x + \partial/\partial y + \partial/\partial z$$

Richard used Darcy's law (1856) to describe the fluid flow and the continuity equation to develop the following equation:

$$\nabla q = - \gamma_d (\partial\theta/\partial t) \quad (16)$$

where

∇q = divergence of the flow,

γ_d = dry unit weight,

θ = volumetric moisture content, and

$\partial\theta/\partial t$ = rate of change of moisture content.

Richards then related the soil suction changes to the moisture variations.

$$\partial\theta/\partial\psi = C_c = \text{capillary capacity} \qquad (17)$$

Combining equations 17 and 18 with Darcy's law, and extending to three dimensions, the following flow expression was obtained:

$$K \nabla^2 \psi + (\partial K_x/\partial x)\ (\partial\psi/\partial x) + (\partial K_y/\partial y)\ (\partial\psi/\partial y) + g\ (\partial K_z/\partial z)\ (\partial\psi.\partial z) = -\gamma_d * A\ (\partial\psi/\partial t) \qquad (18)$$

where

K = hydraulic conductivity,

ψ = total potential, and

$-\gamma_d * A\ \partial\psi/\partial t)$ = rate of volume change of fluid.

Philip and de Vries (1957) combined the equations of liquid flow and vapor flow into the following equation:

$$\partial\theta/\partial t = \nabla\ (D_T \nabla T) + \nabla\ (D_\theta \nabla\theta) + \partial K/\partial z \qquad (19)$$

where

$D_T = D_{Tliq} + D_{Tvap}$ = thermal moisture diffusivity and

$D_\theta = D_{\theta liq} + D_{\theta vap}$ = isothermal moisture diffusivity.

Blight (1971) suggested that Fick's law represented gas transport better than did Darcy's law. The diffusivity in Fick's law (D) which relates mass flux ($\partial m/\partial t$) and pressure

gradient ($\partial P/\partial z$) is a constant. On the other hand, the conductivity relating velocity and pressure gradient varies with the pressure gradient. Fick's law can be stated as

$$\partial m/\partial t = - D \ (\partial P/\partial z) \tag{20}$$

Philip (1969) stated Darcy's law as

$$v = - K(\theta) * \forall\phi \tag{21}$$

where

v = vector flow velocity,

$K(\theta)$ = conductivity, a function of θ,

ϕ = total potential = $\psi(\theta) + Z$,

$\psi(\theta)$ = capillary pressure potential, a function of θ, and

θ = volumetric water content.

He combined the continuity equation

$$\partial\theta/\partial t = - \forall \ v \tag{22}$$

with Darcy's law to write the following diffusion equation:

$$\partial\theta/\partial t = \forall \ . \ (K \ \forall\phi) + \partial K/\partial z \tag{23}$$

Defining the diffusivity $D = K\ (\partial\psi/\partial\theta)$, Philip rewrote equation (20) as follows:

$$\partial\theta/\partial t = \forall\ .\ (D\ \forall\theta) + (\partial K/\partial\theta)\ (\partial\theta/\partial z) \qquad (24)$$

The diffusivity (D) is analogous to the coefficient of consolidation Cv in the consolidation equation.

Bear (1979) separated partially saturated flow into three ranges:

1. Pendular saturation at very low saturation levels leads to almost no flow or pressure transfer.

2. Equilibrium water saturation or the funicular saturation at which both the soil air and the soil water are continuous.

3. Insular saturation, high saturation levels at which the air phase is no longer continuous.

Bear defined the piezometric head in both the saturated and the partially saturated zones as total potential, including both a gravity term and a pore water pressure term, as

$$\phi = z + \psi \qquad (25)$$

where

$\psi = P_w/\gamma_w$ for saturated soil,

$\psi = P_c/\gamma_w$ for partially saturated soil,

z = Elevation head (potential),

P_w = Pore water pressure,

γ_w = Unit weight of soil, and

P_c = capillary pressure

Mitchell (1976) discussed the validity of the Kozeny-Carmen equation for partially saturated soil (Kozeny 1927, Carmen 1956):

$$k = K\ (\mu/\gamma_p) = [(C_s * V_s^2)/S_o^2] * [e^3/(1+e)] * s^3 \qquad (26)$$

where

k = permeability,

K = hydraulic conductivity,

μ = viscosity of the permeant,

γ_p = unit weight of the permeant,

C_s = pore shape factor,

V_s = volume of solid,

e = void ratio,

s = degree of saturation, and

S_o = specific surface per unit volume of particles.

Although this equation works well for the description of conductivity in uniformly graded sands and some silts, serious discrepancies are found in clays. The major factor responsible for failure of the equation in clays is that

the fabrics of such materials do not contain uniform pore sizes. Particles are grouped in clusters or aggregates that result in large intercluster pores and small intracluster pores.

Measurement of matric suction

Matric suction determination is useful in analyzing fluid flow through partially saturated soils. Measurements of suction can be made by several techniques as shown in Table 1.

Soil suction potential is often measured as a negative water head. The absolute value of the logarithm to base ten of suction heads in centimeters is defined as the "P^F" value, a common expression of soil suction. One atmosphere of suction is approximately equal to a "P^F" value of 3, a suction head equal to 10^3 centimeters of water. The logarithmic unit P^F is preferred because most of the soil behavior is linearly related to suction in P^F units. Qualitatively, a P^F value of about 2 corresponds to a very wet condition, 3.5 P^F corresponds to the plastic limit, and a value of approximately 6 P^F is the driest condition for soil.

The following is a summary of the most used techniques of measuring soil suction (Mitchell 1976, McKeen 1988, Kohnke 1968):

1. Piezometers. Water in the piezometer communicates with the soil through a porous stone or filter. Pressures

are determined from the water level in a standpipe, by a manometer, by a pressure gauge, or by an electronic pressure transducer. A piezometer used to measure pressures less than atmospheric is usually termed a tensiometer. Piezometers are often used to measure positive pore water pressures.

2. Gypsum block. The electrical resistance across a gypsum block is measured. The water held by the gypsum block determines the resistance, and the suction in the surrounding soil controls the amount of moisture in the gypsum block. The gypsum block technique is used for measurements of pore pressures less than atmospheric (Kohnke 1968).

3. Pressure-membrane devices. An exposed soil sample is placed in a membrane or a ceramic plate in a sealed chamber. Air pressure in the chamber is used to push water from the pores of the soil through the membrane. The relationship between soil water content and applied pressure is used to establish the relationship between soil suction and water content. The applied pressure at a given water content is taken as the soil suction for that same water content.

4. Consolidation tests. The consolidation stress applied to a sample is taken as the soil water suction when the sample is in "equilibrium" with respect to fluid flow. If the consolidation pressure were instantaneously removed, then a negative water pressure of the same magnitude would be needed to prevent water movement.

5. Vapor pressure methods. The relationship between relative humidity and water content is used to establish the relationship between soil water content and soil suction. The soil is allowed to come to equilibrium with an atmosphere of known relative humidity in a sealed constant-temperature room or container. The relative humidity may be controlled by a solution having a concentration of 3.3% of sulfuric acid (H_2SO_4) in water, whose aqueous vapor pressure corresponds to 98% relative humidity, or P^F 4.5. Figure 14 (McKeen 1988) shows various scales for reporting suction values. The disadvantages of using a dilute solution for this purpose is that its concentration may change during the determination because water is given off or received from the soil sample. Therefore, the concentration of the H_2SO_4 has to be checked and adjusted. More recently, saturated salt solutions have been used for establishing more stable vapor pressure levels for determining the relationship between soil suction and soil water content in the dry range.

The saturated salt solutions have the advantage that their vapor pressure remains the same as long as the solutions are in equilibrium with the solid phase, provided that the temperature remains constant. Change of soil water content does not alter the vapor pressure of such a solution as long as part of the solid phase of the salt is remaining. Table 2 shows five examples of saturated salt solutions used

to obtain water vapor tensions at a temperature equal to 25°C (Kohnke 1968).

The United States Geological Survey (McQueen and Miller 1968) developed a filter paper method for measurement of suction on field gathered samples which were returned to the laboratory for evaluation. The method employs a filter water content versus relative humidity curve, which has been calibrated using salt solution. The filter paper is placed with the soil sample in a temperature controlled closed container for at least a seven-day period for the purpose of reaching equilibrium. The water content of the filter paper and the soil are measured, and the suction is inferred using the calibration curves as shown in Fig. 15 (McQeen and Miller 1968). The advantage of the filter paper method is that it is theoretically applicable over a very wide range of suction values.

6. Freezing-point-depression method. From saturation to a total tension of about 2 or 3 atmospheres, the freezing point of water changes very little. From a tension of 3 to about 25 atmospheres, there is a pronounced change of the freezing point. Beyond this level, there is so little water in the soil that it becomes practically impossible to determine its freezing point. Therefore, the best range to determine total tension by the freezing-point-depression method is from P^F 3.5 to 4.4.

7. Centrifuge. The centrifuge can be used to determine the amount of soil moisture retained against particular centrifuge forces. Briggs and McLane (1907, 1910) have developed a technique in which a wet sample of soil is subjected to a centrifugal force 1000 times the force of gravity for 40 minutes. The resultant water content is called the moisture equivalent (similar to "field capacity"). In this centrifuge test, the results are only used to provide qualitative data for comparisons of suction between various soil types (Kohnke 1968).

8. Thermocouple psychrometer. A psychrometer is defined as two similar thermometers with the bulb of one being kept wet so that the loss of heat that results from evaporation causes it to register a lower temperature than the dry thermometer; the difference between the two temperature readings represents a measure of the dryness of the atmosphere and is called the wet bulb depression. From this information, the relative humidity can be computed. For more details and discussion, refer to McKeen (1988).

Factors Affecting the Prediction of Saturated Hydraulic Conductivity of Clay Liners

Several investigators have addressed the influence of various factors on the measurement of the saturated hydraulic conductivity of compacted clays both in the laboratory and in-situ (Acar et al. 1987, Bagchi et al. 1985, Berystorm 1985, Bogardi et al. 1989, Boynton and Daniel 1985, Carpenter

and Stephenson 1986, Daniel 1984, Elzeftawy and Cartwright 1979, Gorden et al. 1989, Korfiatis et al. 1987, Mitchell and Younger 1966, Mitchell 1976, Oakley 1987, Olsen et al. 1979, Peirce et al. 1987(a), Schmid 1966, Siva et al. 1979, Stewart and Nolan 1987, Taylor 1948, Wit 1966). Therefore, the factors affecting the prediction of saturated hydraulic conductivity will be separated into laboratory and field factors, and each will be briefly reviewed.

Laboratory factors

Several investigators have studied the many factors that affect the measurement of the saturated hydraulic conductivity of compacted clays in the laboratory. Broadly speaking, the factors influencing hydraulic conductivity can be classified into three categories.

1. Testing apparatus factors. These factors are associated with testing variables such as type of permeameter, confining pressure, direction of flow, and hydraulic gradient. The three most common types of permeameters are the consolidation cell, rigid wall, and flexible wall. These permeameters were discussed previously.

a. Type of permeameter. Boynton and Daniel (1985) have outlined qualitatively the difference in some parameters when using the three type of permeameters. This outline is shown in Table 3. Figure 16 (Boynton and Daniel 1985) shows the results of two types of clays tested using the three

different permeameters. Based on these results, it is concluded that the type of permeameter did not have a large effect on the measured hydraulic conductivity; the differences in the values of conductivity were substantially less than one order of magnitude; and no one type of permeameter consistently yielded higher or lower values than the other types. However, Stewart and Nolan (1987) have found that the conductivity measured from the rigid wall permeameter is consistently lower than the other types as it is shown in Fig. 17 (Stewart 1987).

b. Confining pressure. This factor affects the hydraulic conductivity measured by the flexible wall permeameter only since the other types do not apply an all-around confining pressure to the tested sample. This is done in order to prevent side wall leakage and facilitate sample saturation. Figure 18 (Boynton and Daniel 1985) shows that as the confining pressure increases, the conductivity decreases. Korfiatis et al. (1987) have shown that the conductivity value decreased twice as much as that reported by Boynton and Daniel for the same increase in confining pressure.

c. Direction of flow. In all laboratory conductivity tests the flow is restricted to the vertical direction. This is because it is easier and better simulates the flow in the field. Also for compacted soils, the lateral flow is the same as the vertical flow.

d. Hydraulic gradient. Mitchell and Younger (1966) have shown that for clays, tested in flexible wall permeameter, at low hydraulic gradient, the hydraulic conductivity tends to be very low and the flow deviates from equation 1. They found that this phenomenon exists due to dislodging and washing down of fine particles in samples with low initial compaction density. Mitchell and Younger also showed that samples tested under increasing hydraulic gradient have lower hydraulic conductivity than a decreasing one. Olsen and Daniel (1979) has reported some studies which showed that as hydraulic gradient increases so did the predicted conductivity by 5 to 84 times.

2. Permeant factors. These factors are associated with the type and properties of the permeant. When hydraulic conductivity is mentioned, it is understood that water conductivity is referred to. There are two main water properties that can affect the speed of water flow through soils.

a. Viscosity and density. The relationship between viscosity and density of water with the conductivity is given in the well-known Kozeny-Carman equation 26, and it can be rewritten as

$$K = k \ (\gamma_p/\mu) \tag{27}$$

where

K = Hydraylic conductivities,

k = Permeability,

γ_p = Unit weight of permeant (water), and

μ = Viscosity of permeant.

Equation 27 suggests that the conductivity varies directly with the density and inversely with the viscosity of percolating water (or any other fluid). The density and the viscosity terms are usually taken as constant and equal to one for water at laboratory temperature.

b. Normal and deaired water. Hydraulic conductivity was thought to be less when using normal (tap) water because a greater number of flow channels could become blocked by evolved air bubbles than when using deaired water. The opposite was found (Stewart and Nolan 1987).

3. Soil factors. These factors associated with physical and chemical characteristics of the soil. Furthermore, these factors affect the measured conductivity differently for different soils. Soil properties by far have the largest influence on the predicted conductivity.

a. Molding water content and degree of saturation. Darcy's law and other relations for predicting the conductivity have been developed or experimentally established on soils with 100% saturation. Conductivity is greatly affected if air, even in small amounts, remains in

the pores of soil. Conductivity drops to very low values at degree of saturation less than 75% (Sing 1967). Figure 19 (Mitchell 1976) shows that as the degree of saturation increases so does the conductivity for compacted clays tested in flexible wall permeameter. Most of the time, it is easier to obtain and more accurate to relate the conductivity to the molding water content instead of degree of saturation. Both Fig. 16 and Fig. 17 show a plot of conductivity versus molding water content, and it can be seen that as the molding water content increases, the conductivity decreases up to a maximum (optimum) value. Beyond this optimum value a further increase in the molding water content will result in an increase in the conductivity. This can be explained by the fact that at lower molding water content (or lower degree of saturation) the water flows through the soil under both the hydraulic head and suction head. As the soil becomes saturated, most of the air will be driven out of the soil, the suction head will be minimal, the water will flow under the hydraulic head only, and will result in the lowest conductivity value. Beyond the lowest conductivity an increase in water content will result in a change of soil fabric from a semidispersed to a fully dispersed structure which possess higher conductivity.

b. Dry unit weight of soil. The relationship between the conductivity and the dry unit weight of soil is shown in Fig. 16. At low molding water content and dry unit

weight the fabric structure of the soil is mainly flocculated (possesses a high degree of porosity or void ratio), and the conductivity is highest. As the molding water content increases, the dry unit weight increases, the degree of porosity or void ratio decreases, the soil structure changes gradually from fully flocculated to semiflocculated, and this results in a decrease in the conductivity value. At and around the optimum molding water content, the dry unit weight is maximum, the soil structure tends to be semidispersed, and the conductivity is lowest. At a molding water content greater than this region, additional water tends to force soil particles apart, changing the soil structure to near fully dispersive. This will lead to a high degree of porosity or void ratio and, therefore, a lower dry unit weight and higher conductivity value.

c. Sample diameter. Boynton and Daniel (1985) have studied the effect of sample diameter of fire clays tested in flexible wall permeameter and obtained the plot shown in Fig. 20. He concluded that the measured conductivity was essentially independent of sample diameter and that the conductivity of the largest sample used was one third to twice the value measured on the smallest samples. However, the larger the sample diameter, the more likely the sample will contain more hydraulic defects and the closer the sample will be in resembling the field conditions.

d. Adsorbed water. The adsorbed water surrounding the fine-grained soil particles is not free to move, and, hence, it causes an obstruction to the flow of free water by reducing the effective pore space available for the passage of water. It is difficult to define the pore space occupied by adsorbed water in a soil. According to a crude approximation after Casagrande, 0.1 may be taken as the voids ratio occupied by adsorbed water, and the conductivity may roughly be assumed to be proportional to the square of the net void ratio of $(e - 0.1)^2$. Adsorbed water has a marked influence on the conductivity of clays. In a laboratory, it is normal to use a high gradient for testing clays, but in actual field problems, the hydraulic gradient is much less. There is a hydraulic gradient (threshold gradient) for clays at which the conductivity is essentially zero. Lambe and Whitman (1979) reported that this gradient for some clays is equal to 20 to 30. Mitchell (1976) suggested that the value of threshold gradient could be higher for montmorillonite clays and reported a maximum value of 900.

e. Mini-aging. Figure 19 (Mitchell 1976) shows the conductivity of clay samples aged for 21 days and tested in flexible permeameter. Aged samples did not display consistently higher or lower conductivity than the unaged samples. The same conclusion is reached by Boynton and Daniel (1985) after testing different clays in exactly the same way as it is shown in Fig. 21. Olson and Daniel (1979)

have suggested that prolonged conductivity tests (and probably aging) may result in a substantial reduction in conductivity due to clogging of the flow channels by organic matter that grows in the soil during the test (and may be during aging too).

f. Direction of flow. Lambe and Whitman (1979) suggested that compacted clays are flocculated dry of optimum, resulting in a lower degree of hydraulic anisotropy, and dispersed wet of optimum, resulting in higher degree of hydraulic anisotropy. Olsen and Daniel (1979) suggested that clods of clay are hard when the molding water content is dry of optimum, resulting in large interclod void space, and soft when they are wet of optimum, resulting in minimal interclod void space. In this case, the only source of anisotropy would be the flattening of clods during compaction. Boynton and Daniel (1985) have used flexible wall permeameter to test compacted clays that were sampled in horizontal and vertical directions. He concluded that soil fabric has no discernable effect on hydraulic anisotropy.

g. Desiccation. Literally no data were found on desiccation cracking in compacted clays and its influence on hydraulic conductivity. Boynton and Daniel (1985) prepared 2.5-inch thick compacted clay slabs and found a 1 millimeter wide crack appeared after 4 hours, and the crack penetrated the slab after 8 hours. The cracked clays were then sampled and tested in a flexible wall permeameter under different

confining pressures. The result is shown in Fig. 18. It was concluded that desiccation cracks can penetrate compacted clay to a depth of several inches in just a few hours. Furthermore, the cracks tend to close when moistened and the hydraulic conductivity is not affected by a large amount.

h. Sample height. Sample height was studied by Korfiatis et al. (1987). In this study, he tested compacted clays in a flexible wall permeameter and followed an orthodox procedure. He tested a sample 3 inches thick and 2.5 inches in diameter. Then, the same sample was divided into two halves and tested, and the same two halves were divided into four equal pieces and also tested. He concluded that the hydraulic conductivity increases with increasing sample height as shown in Fig. 22.

i. Amount and type of clays. Little data exist on the effect of the amount and type of clays on the measured hydraulic conductivity. Mitchell (1976) tested compacted clays in flexible wall permeameter and found that increasing amounts of clay from 5% to 15% led to a decrease in conductivity by four times. Daniel (1987) has tested compacted clays with different plasticity indices in a flexible wall permeameter and found generally that as the plasticity index increases, the measured hydraulic conductivity decreases. This is shown in Fig. 23.

j. Termination criteria. This factor deals with the amount of outflow of water from the tested sample

necessary to assume that a steady state value of hydraulic conductivity has been reached. This amount is usually expressed in terms of the total volume of pores. Peirce and Witter (1986) has studied this factor on compacted clays in a flexible wall permeameter and concluded that about one-half of the pore volume is necessary to reach a steady state conductivity. This is shown in Fig. 24.

Field factors

The factors that affect the field measurement of hydraulic conductivity of clays are many and are very difficult to quantify and measure, each of which tends to have large influence on the measured conductivity. Olsen and Daniel (1979) stated, "Field testing for measurement of conductivity in unsaturated soils is at such a elementary stage of development that field measurements cannot be recommended" (p. 55). Field conductivity testing is still at a rudimentary stage and still not performed even in large landfill projects. Some of the suggested methods for field infiltration tests are shown in Figs. 10 and 11. However, in addition to the permeant and soil factors mentioned above, there are other factors to be considered. These are

1. Homogeneity and isotropy. These factors affect the field conductivity more than the laboratory conductivity. This is because in the field the volume, thickness, placement, and compaction of the clays are much greater and

different than those in the laboratory. Due to the relatively large volume of soils handled in the field, soils might have different amounts and types of clays even if the supply source is the same. This will result in different in-situ densities upon compaction, and different areas might experience different amounts and types of compaction. This will lead to inhomogeneity and anisotropy of the compacted clays.

2. Discontinuities. Field discontinuities in the compacted clays exist as desiccation cracks due to exposure to temperature, areas of low densities due to low compaction, pockets of high sand content and low clay content, zones of contaminated clays with the in-situ sandy soils, and areas with large interclod void space. All these discontinuities will result in an increase of hydraulic conductivity of the compacted clays.

3. Suction and saturation. In the field, both the compacted clays and the sandy subgrade below it are partially saturated soils and, consequently, both possess a certain amount of suction. This suction is very difficult to measure, will increase the hydraulic gradient, and leads to an increase in the infiltration of water through the clays. This is shown in Fig. 25. Many investigators have measured the suction of the clays as a function of the clay moisture content (Daniel et al. 1979, Elzeftawy and Cartwright 1979, Hamilton et al. 1979, Gorden et al. 1989, McKeen 1988, Olsen

and Daniel 1979, Pachepsky and Scherbakov 1984). Figure 26 (Daniel et al. 1979) represents the typical result of such an investigation, and it shows that as the moisture content increases from 7% to 20%, the suction decreases from 43 to 1.5 atmospheres (632.1 to 22.1 psi), respectively. Stewart and Nolan (1987) showed that the distribution of soil saturation after performing the field infiltration tests is not uniform as can be seen in Fig. 27. The figure also shows that the moisture migrated laterally in all the tests by a considerable amount. Stewart also measured the field hydraulic conductivity with time and found it to vary by one-half to one order of magnitude, as can be seen in Fig. 28.

4. Clay thickness. The thickness of the clay liner in the field ranges from 8 inches (top cover) up to 5 feet, while the thickness of the clay sample tested in the laboratory is no greater than 3 inches. The only available data on this factor are shown in Fig. 22 (Korfiatis et al. 1987). This research was performed on a compacted clay sample with a thickness of 3 inches and, therefore, cannot be compared to the field thickness.

Purpose and Scope of this Research Project

The purpose of this research project is to develop a new and rugged methodology of predicting field hydraulic conductivity for compacted natural Floridian clays and to study a number of field and laboratory factors that are

affecting the prediction of hydraulic conductivity. Some of these factors were expressed by the local industry in Florida, and the others were deduced based on the deficiencies observed from the review of previous work related to hydraulic conductivity. Furthermore, this study was designed such that the predicted hydraulic conductivity values are rugged and insensitive, to some degree, to possible mathematical manipulations. The scope of work for this research involved the following:

Bulk Sampling, Properties, and Sample Preparation

A number of bulk soil samples were obtained from the Mid-Florida Mining Corporation's (MFM) clay mine in Ocala, Florida. This clay was, and still is, used in the construction of a number of landfills. It is marketed under the tradename of "Terra-Seal Natural Premix®." One homogenous soil sample was obtained from these samples, and all subsequent laboratory tests were performed using this homogeneous sample. A number of laboratory tests were performed to obtain the index and physical properties of the soil. These properties were compared with those established previously by a local professional testing laboratory. Samples for laboratory conductivity tests were prepared using a 4-inch inside diameter, with variable length, cast acrylic plastic tubing, and in accordance with ASTM D698A and D1557A (ASTM 1989). Undisturbed field samples were obtained using

three different steel sleeve sampling apparatuses designed using some of Hvorslev's (1962) recommendations. These apparatuses were also used to perform field infiltration tests. Undisturbed field samples were also obtained using block sampling techniques.

Laboratory Work

A large number of compacted soil samples were tested in a rigid wall permeameter, and a number of relationships and the influence of various factors on the soil hydraulic conductivity were established. The most important of these relationships is the soil conductivity versus soil suction, versus dry unit weight, versus molding water content. Other factors studied are the effect of sample height, number of layers in the sample, hydraulic gradient, time, drying time, and field sampling. The distribution of moisture content versus depth of a number of samples after conductivity and after drying were also established.

Field Work

Field work was performed at two landfill projects located in Florida. The clay used in the construction of these two projects is from the same source and is the same Terra-Seal Natural Premix used in this study. Two field infiltration tests were performed on the top cover at the Southwest Alachua Landfill located in Archer, Florida. This

landfill was constructed during 1986. Three 10- by 9-foot by 9-inch-thick test strips were constructed, using three different layerings. These test strips were constructed prior to the construction of the second project, Astatula Ash-Residu Monofill landfill located in Astatula (40 miles south of Ocala), Florida. These test strips were used to study the method of construction, desiccation cracks, density and moisture content distribution, and to perform five field infiltration tests. Three additional infiltration tests were performed on the actual landfill after it was constructed.

Prediction and Comparison of Hydraulic Conductivity

The results of the suction tests and the field infiltration test results were used to predict the saturated hydraulic conductivity of the field compacted clays. The predicted values agreed very closely with those obtained in the laboratory by the author and two independent professional testing laboratories. The relationship between laboratory conductivity and the various factors studied were obtained and quantified.

TABLE 1. Methods of Measuring Suction (McKeen 1988)

Technique	Range (pF)	Remarks
Suction Plate	1.0-3.0	Matric
Pressure Plate	1.0-3.0	Matric
Pressure Membrane	0.0-6.2	Matric
Osmotic Cell	2.0-4.1	Total
Centrifuge	3.7-4.1	Matric
Vacuum Desiccator	5.0-7.0	Total
Sorption Balance	5.0-7.0	Total
Thermocouple Psychrometer	2.5-4.8	Total
Filter Paper Method	0.1-7.0	Total
Heat Dissipation in a Ceramic	0.0-4.2	Matric

TABLE 2. Saturated Salt Solution Versus Relative Humidity (Kohnke 1968)

Salt	Relative Humidity at 25°C	pF
$CaSO_4$	97.8	4.49
$NH_4H_2PO_4$	93.0	5.00
NH_4Cl	79.3	5.51
$Mg(NO_3)_2$	52.0	5.96
$KC_2H_3O_2$	19.9	6.36

TABLE 3. Various Parameters for Three Permeameters (Boynton and Daniel 1985)

Test parameter (1)	Type of Permeameter: Compaction mold (2)	Consolidation cell (3)	Flexible wall (4)
Side-wall leakage	Leakage is possible	Applied vertical stress makes leakage unlikely	Leakage is unlikely
Void ratio (e)	Relatively high e because applied vertical stress is zero	Relatively low e because a vertical stress is applied	Relatively low e because an all-round confining pressure is applied
Degree of saturation	Specimen may be unsaturated	Specimen may be unsaturated	Application of back-pressure is likely to cause essentially full saturation
Voids formed during trimming	Impossible; soil is tested in the compaction mold and is not trimmed	Voids may have formed, but application of a vertical stress should help in closing any voids	Voids are not relevant; the flexible membrane tracks the irregular surface of the soil specimen
Portion of sample tested	All of the compacted specimen is tested, including the relatively dense lower portion and the relatively loose upper portion; the dense lower portion may lead to measurement of relatively low k	Only the central portion of the specimen is tested; the upper and lower third of the specimen are trimmed away	One centimeter of soil is trimmed off both ends of the compacted sample

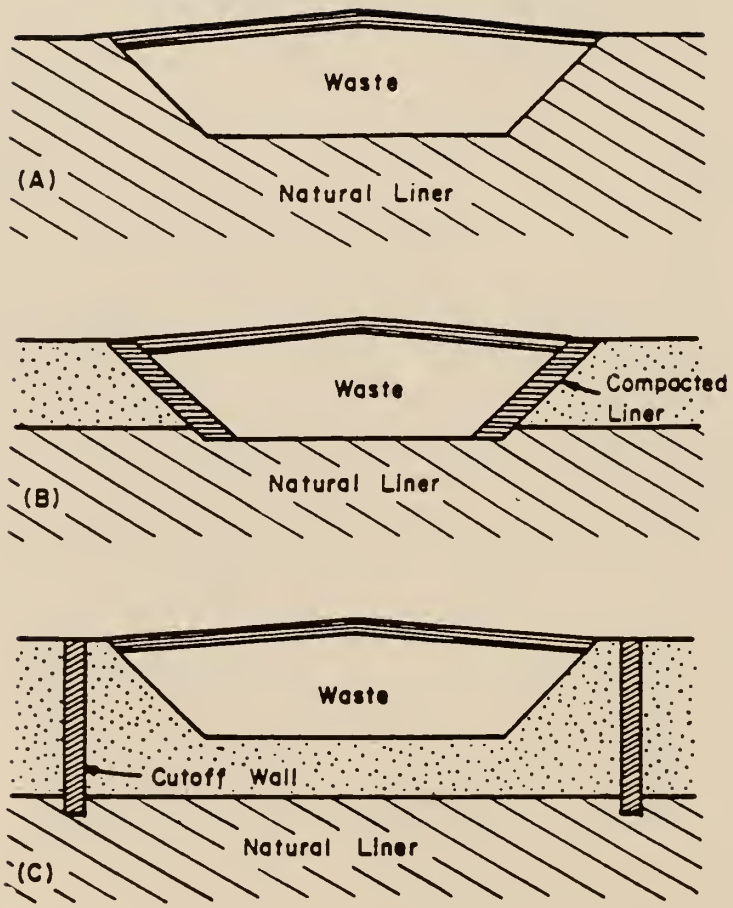


Fig. 1. Examples of Natural Liners (Daniel 1987).

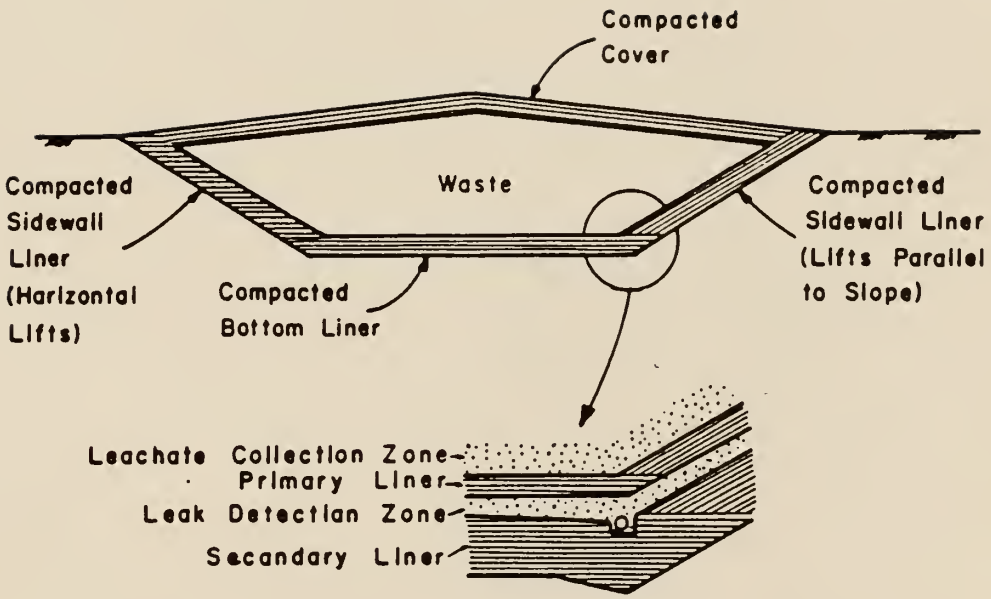


Fig. 2. Types of Compacted Liners (Daniel 1987).

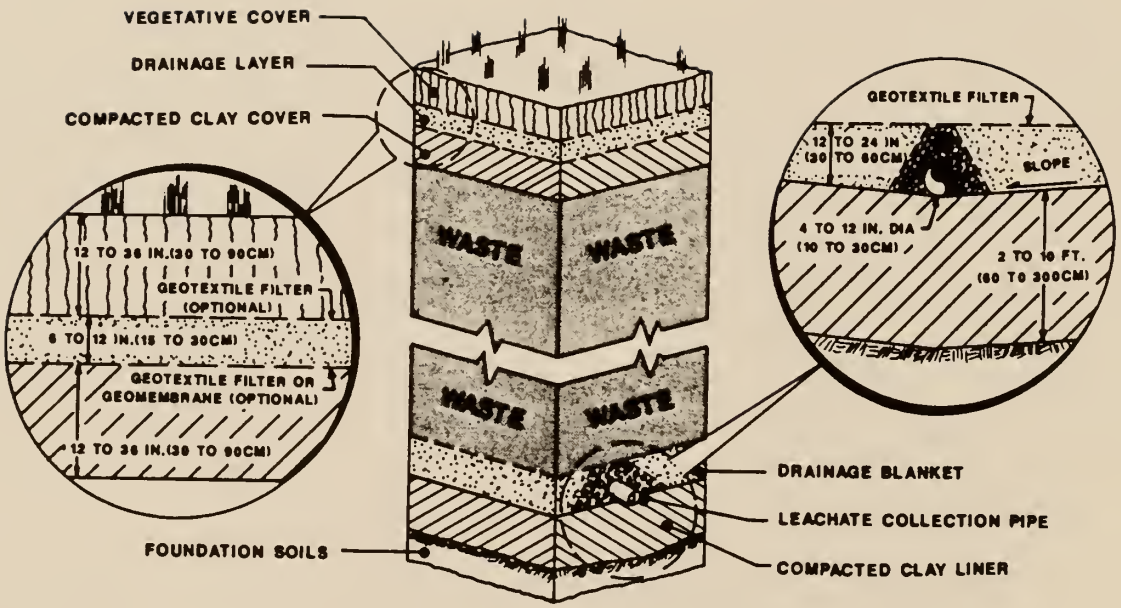


Fig. 3. Typical Landfill Section and Components (Oakely 1987).

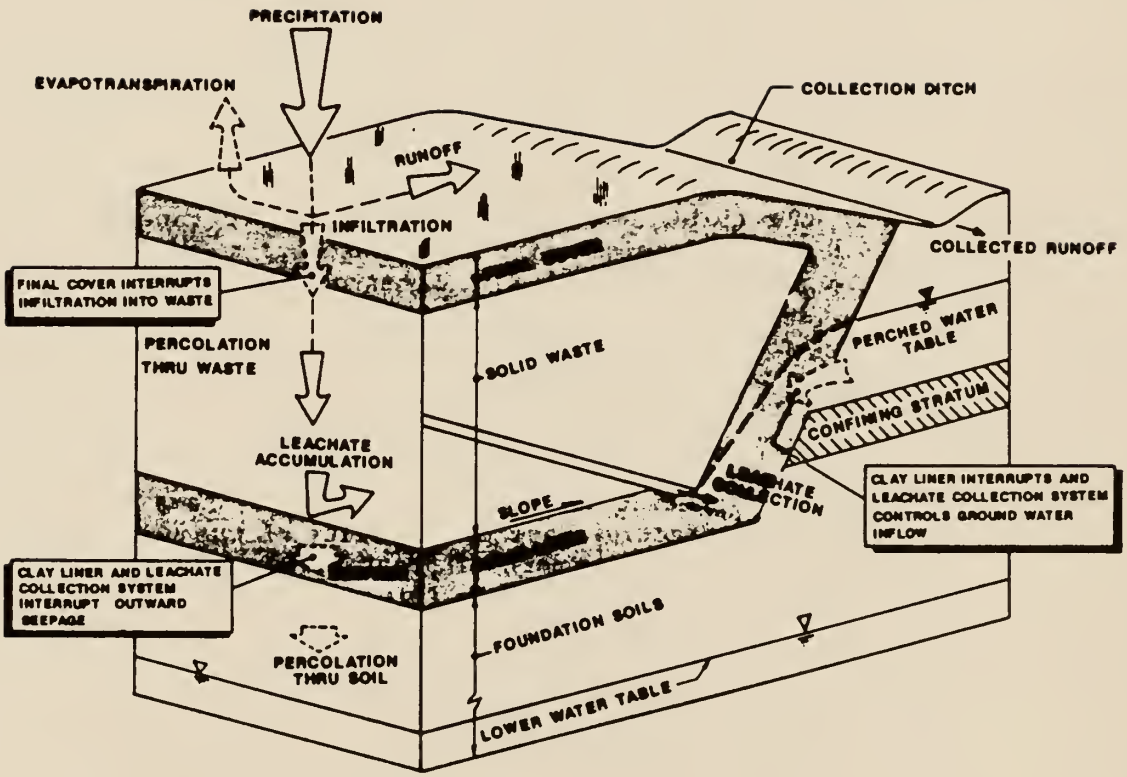


Fig. 4. Hydrological Cycle as Applied to Landfill System (Oakely 1987).

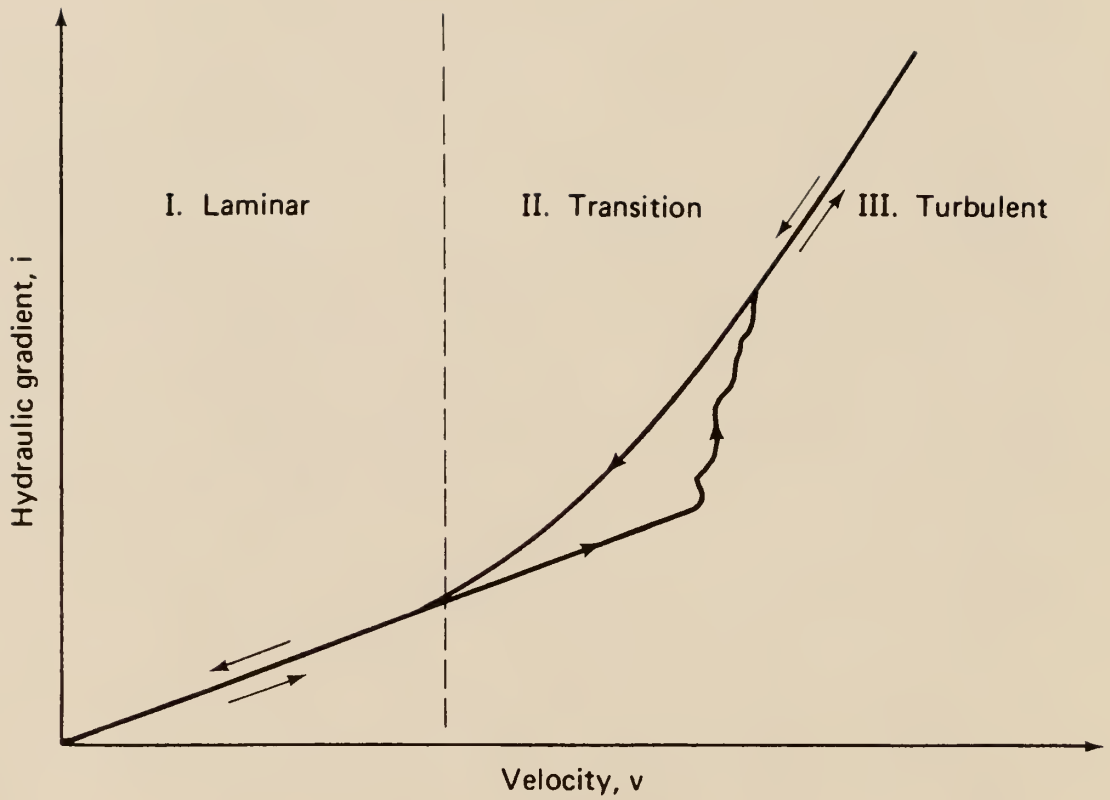


Fig. 5. Zones of Laminar and Turbulent Flow (Taylor 1948).

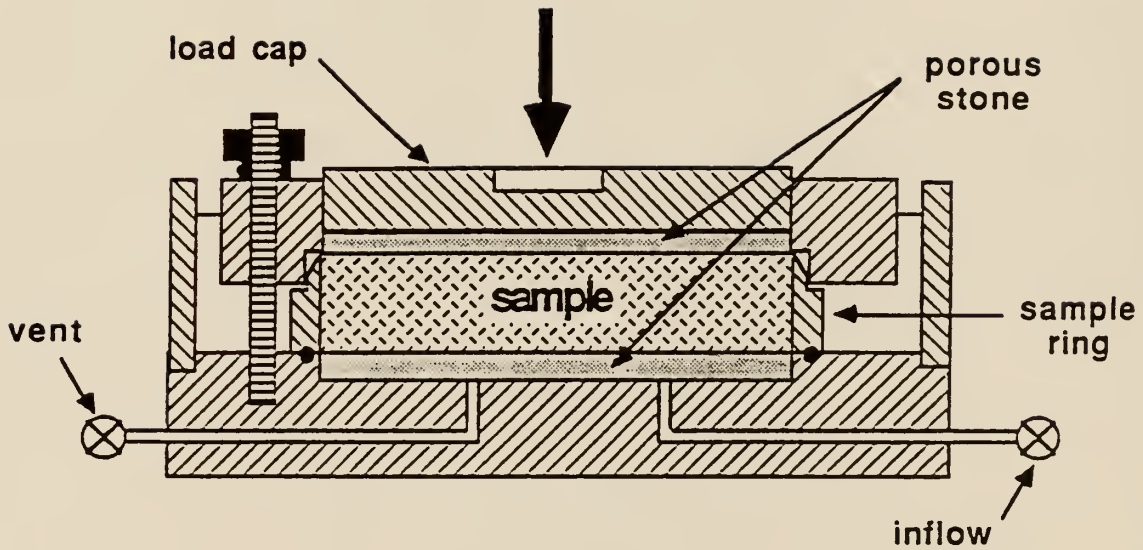


Fig. 6. One-Dimensional Schematic of Consolidation Cell Permeameter.

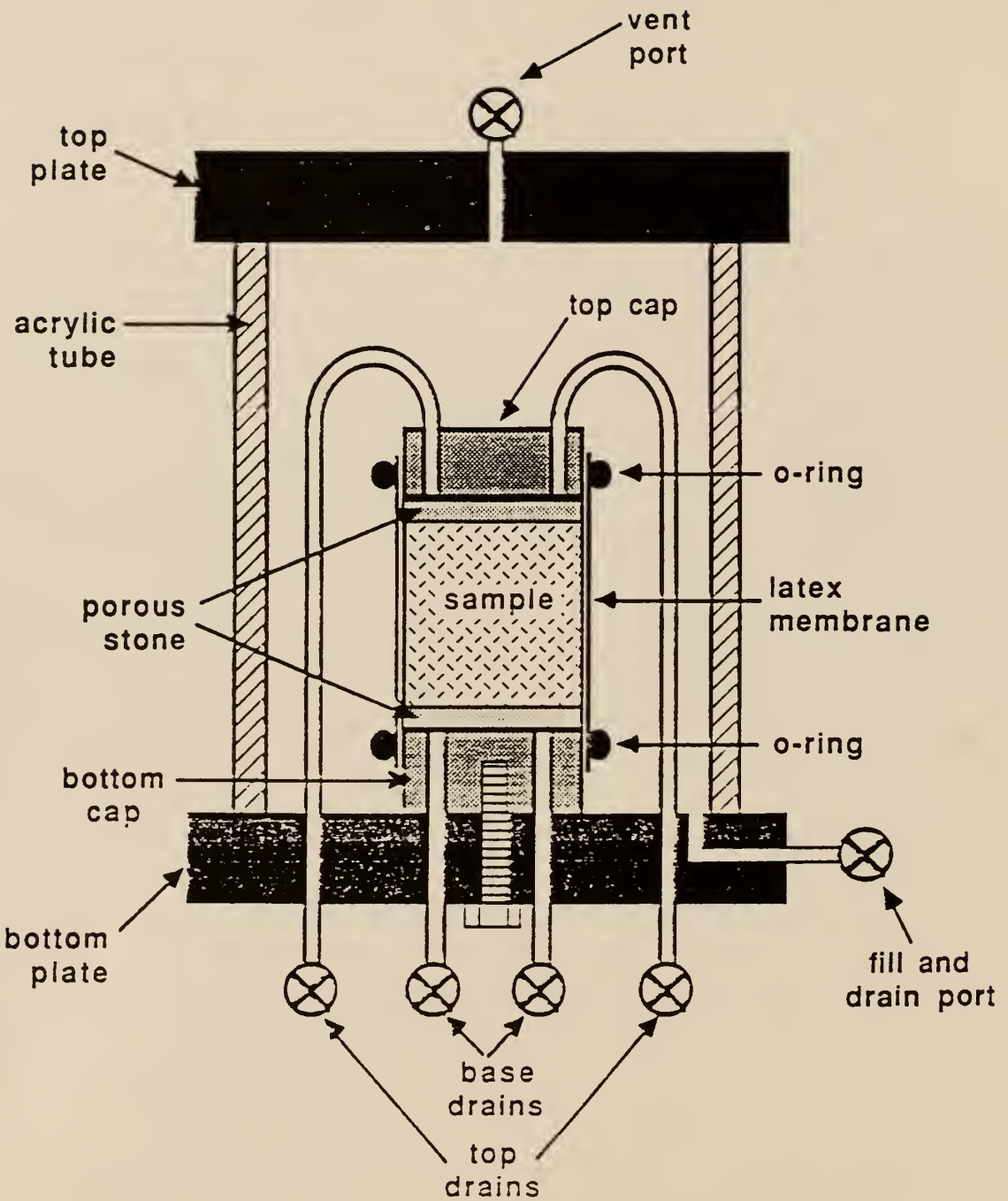


Fig. 7. Schematic of Flexible Wall Permeameter.

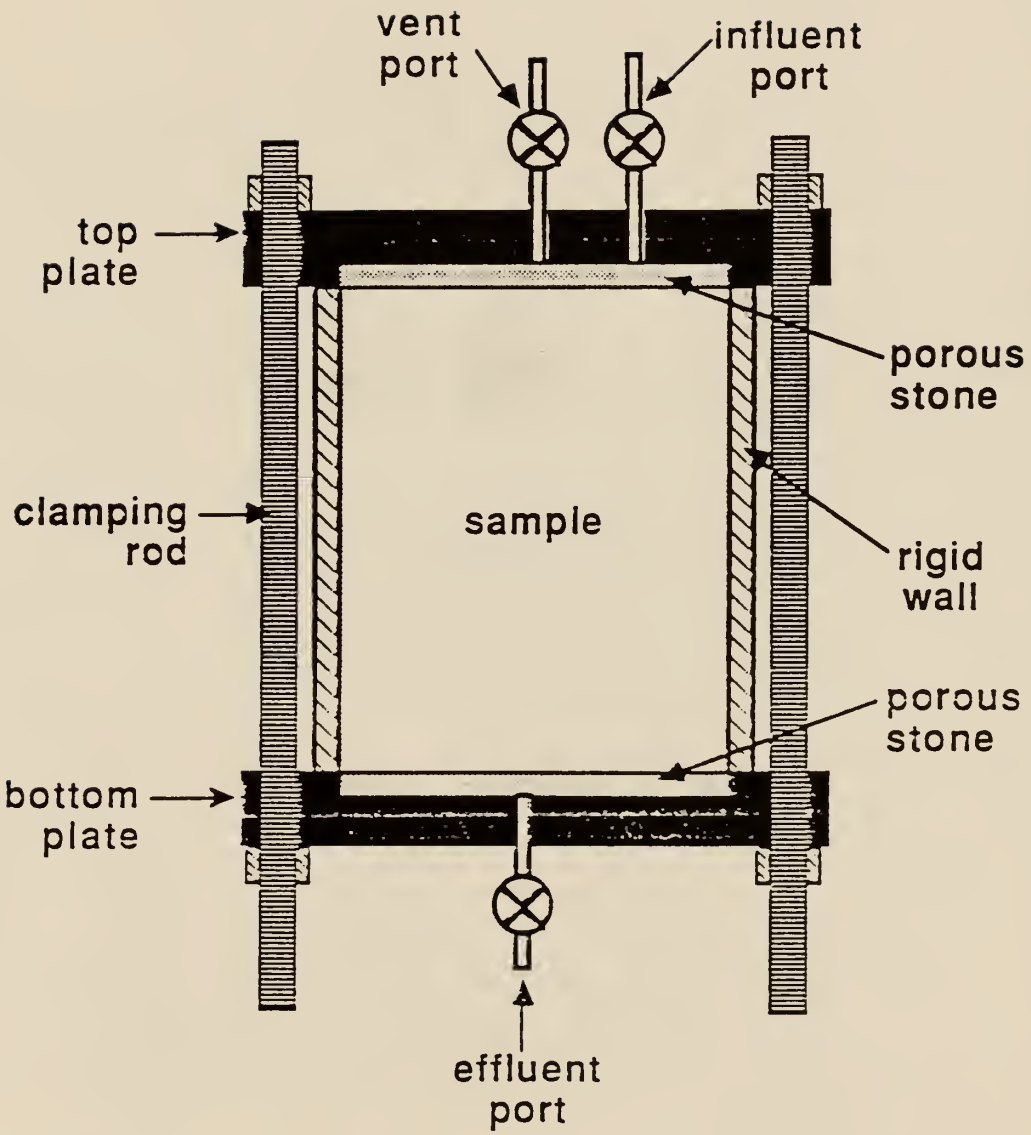


Fig. 8. Schematic of Rigid Wall Permeameter.

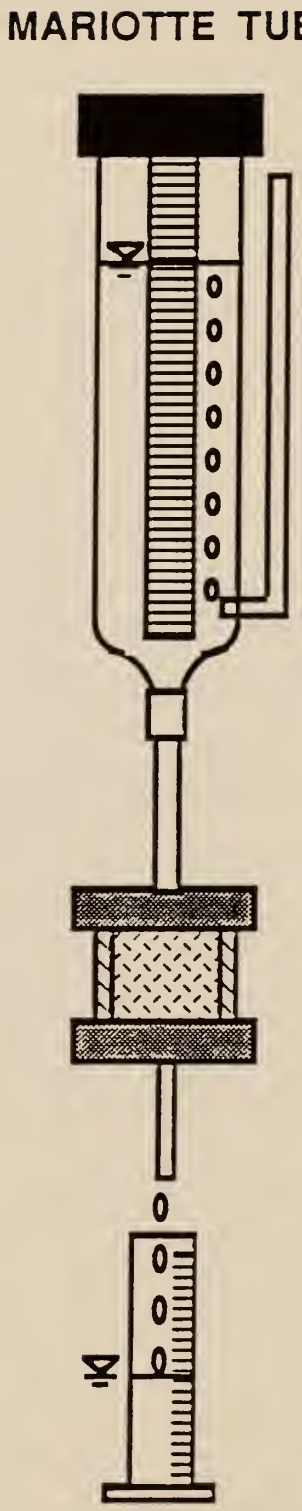
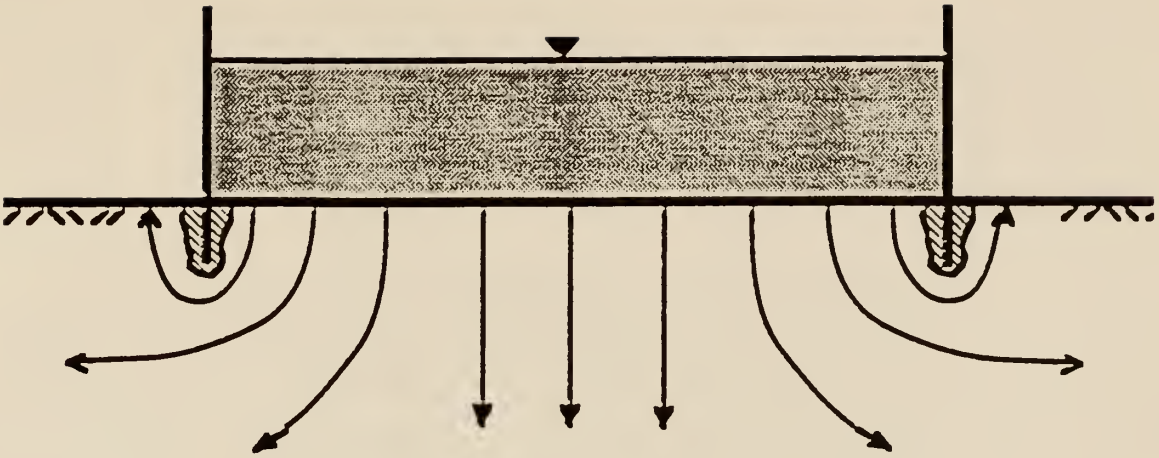
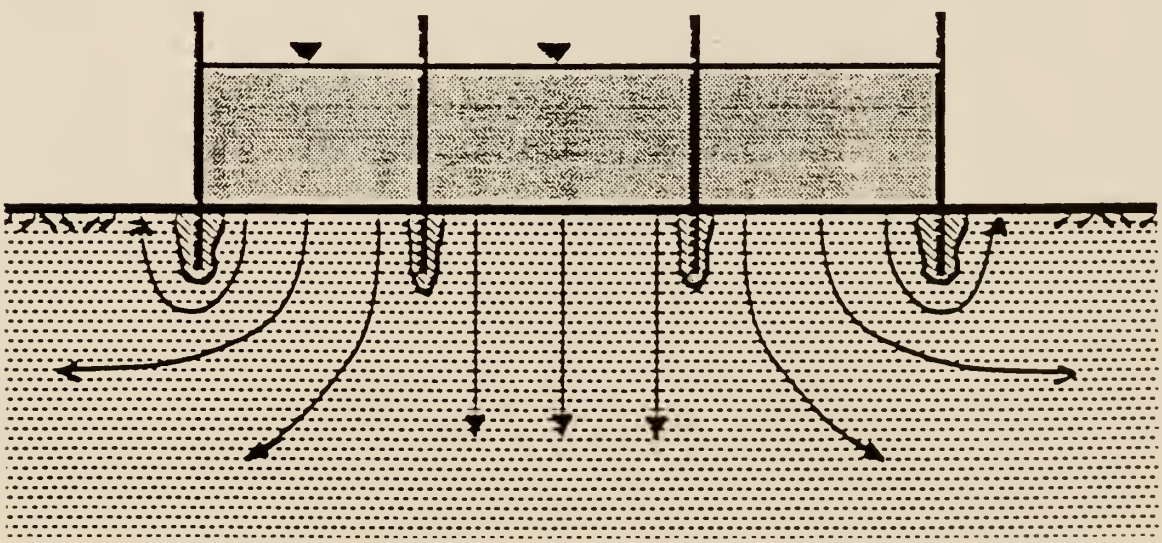


Fig. 9. Schematic of Mariotte Tube.



Open Single-Ring Infiltrometer



Open Double-Ring Infiltrometer

Fig. 10. Schematic of Single and Double Ring Infiltrometers.

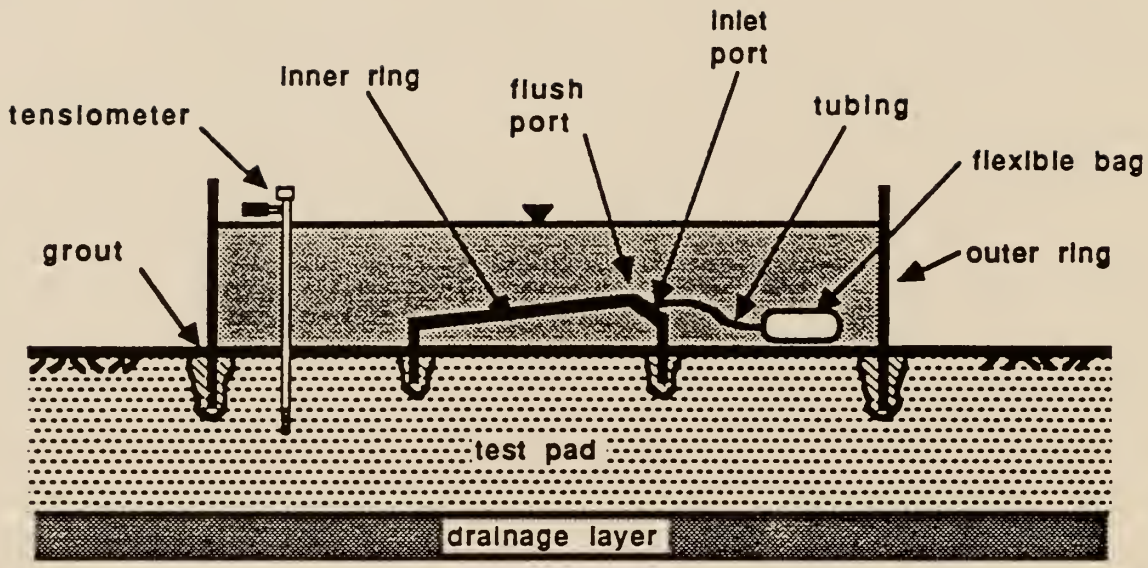


Fig. 11. Schematic of a Sealed-Double Ring Infiltrometer (University of Texas, College of Engineering, 1990).

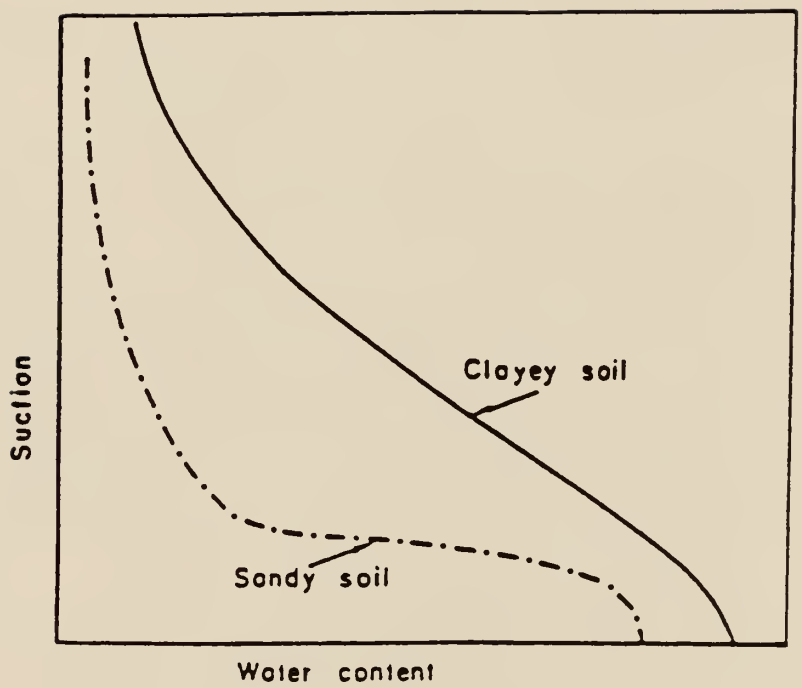


Fig. 12. Soil Suction versus Water Content (Hillel 1971).

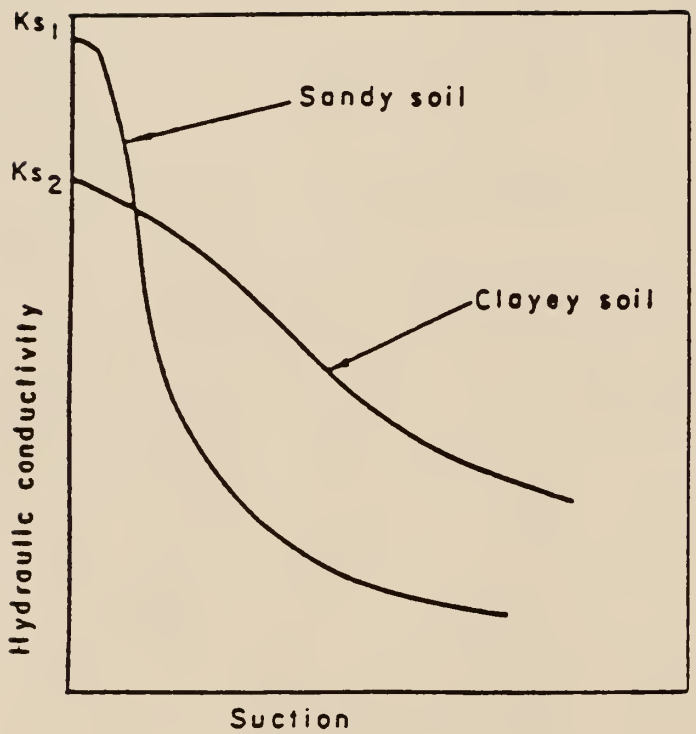


Fig. 13. Soil Suction versus Conductivity (Hillel 1971).

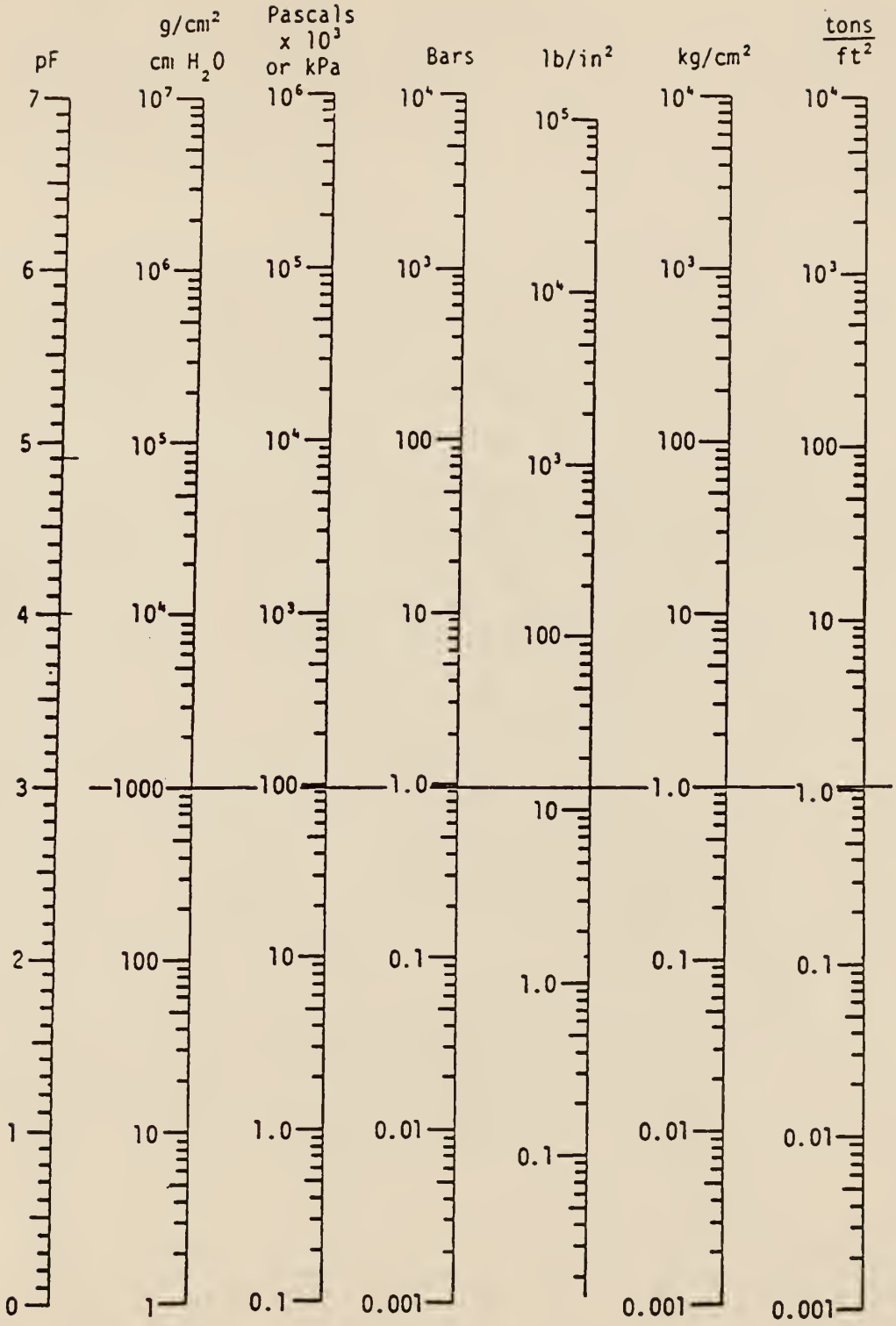


Fig. 14. Scales for Reporting Suction Values (McKeen 1988).

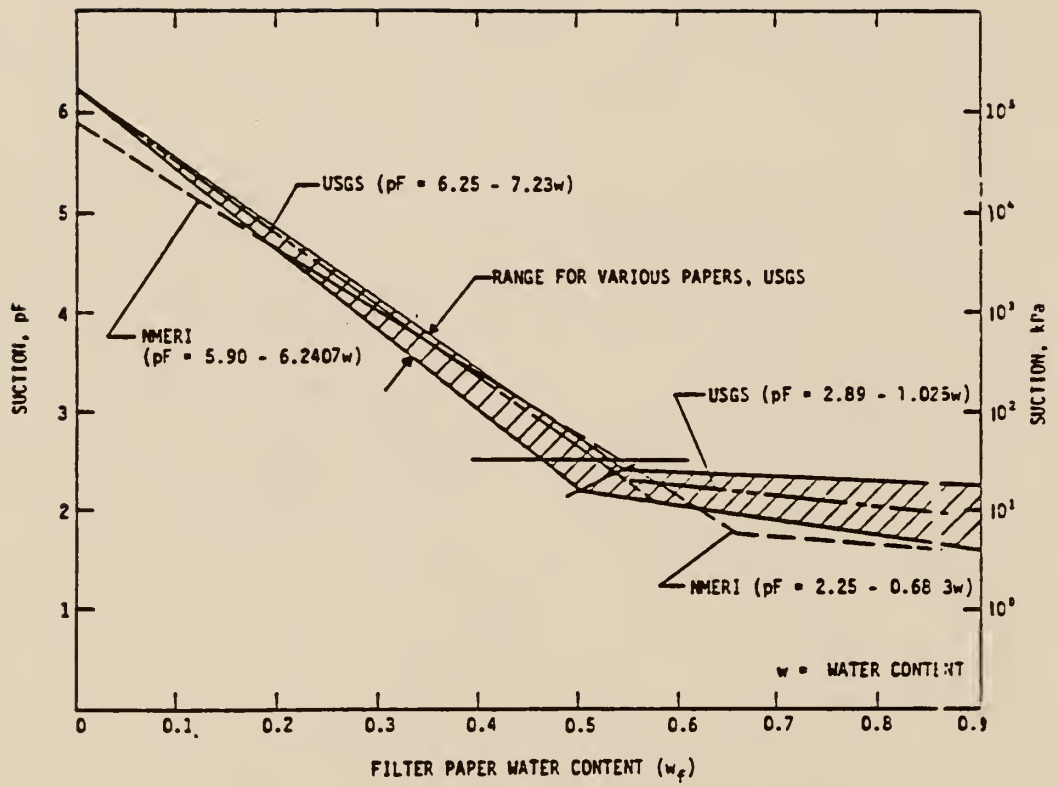


Fig. 15. Filter Paper Calibration Curves (McQeen and Miller 1968).

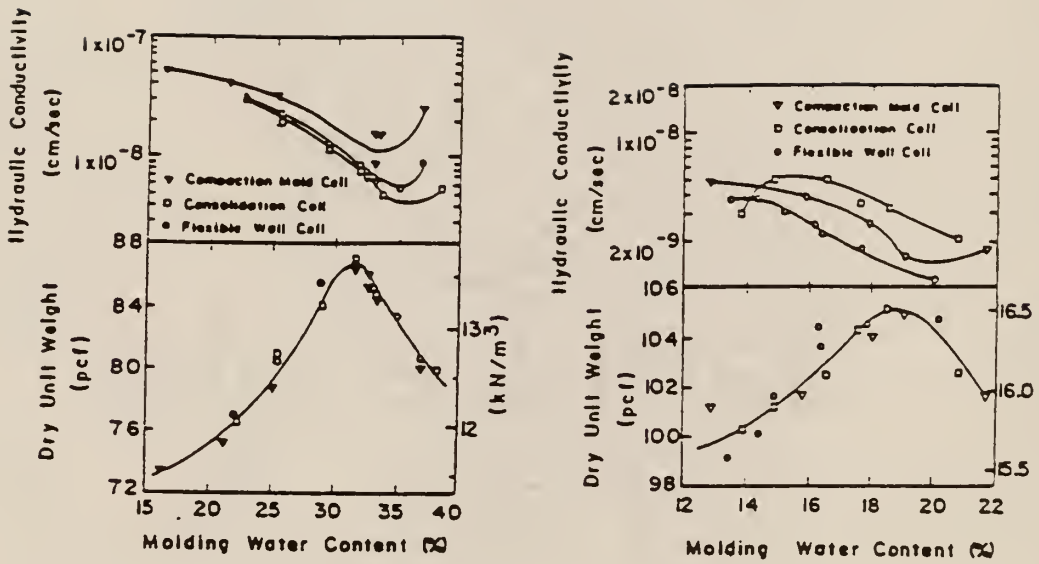


Fig. 16. Conductivity vs. Dry Unit Weight vs. Molding Water Content for Two Different Clays (Boynton and Daniel 1985).

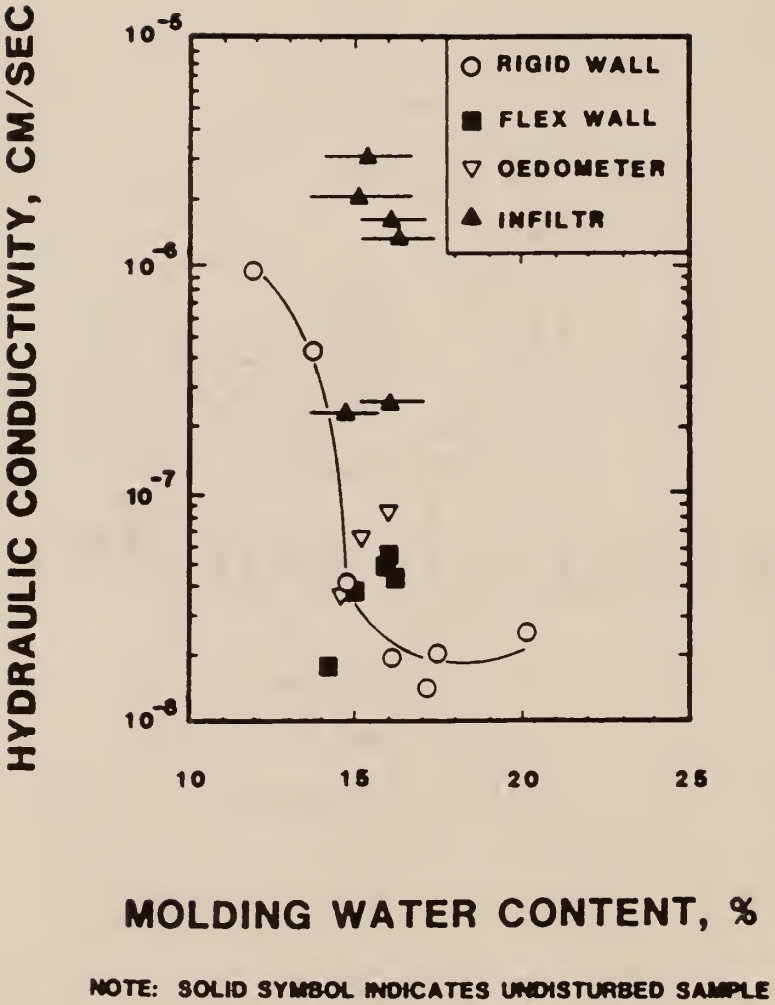


Fig. 17. Summary of Laboratory and Field Infiltration Tests (Stewart and Nolan 1987).

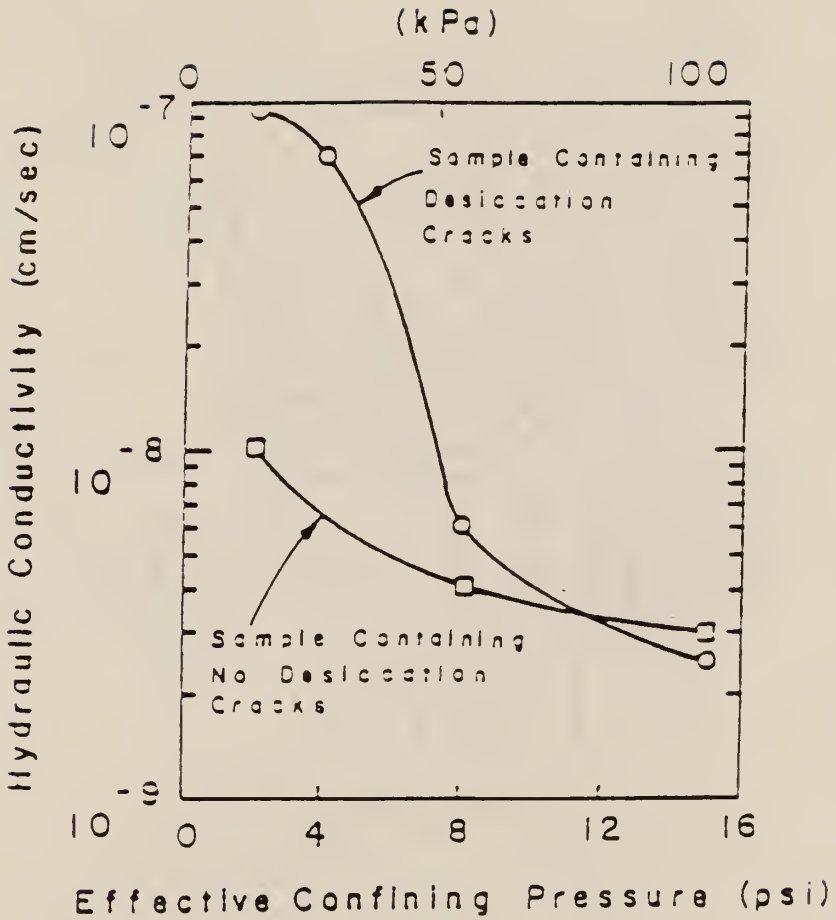


Fig. 18. Conductivity vs. Confining Pressure (Boynton and Daniel 1985).

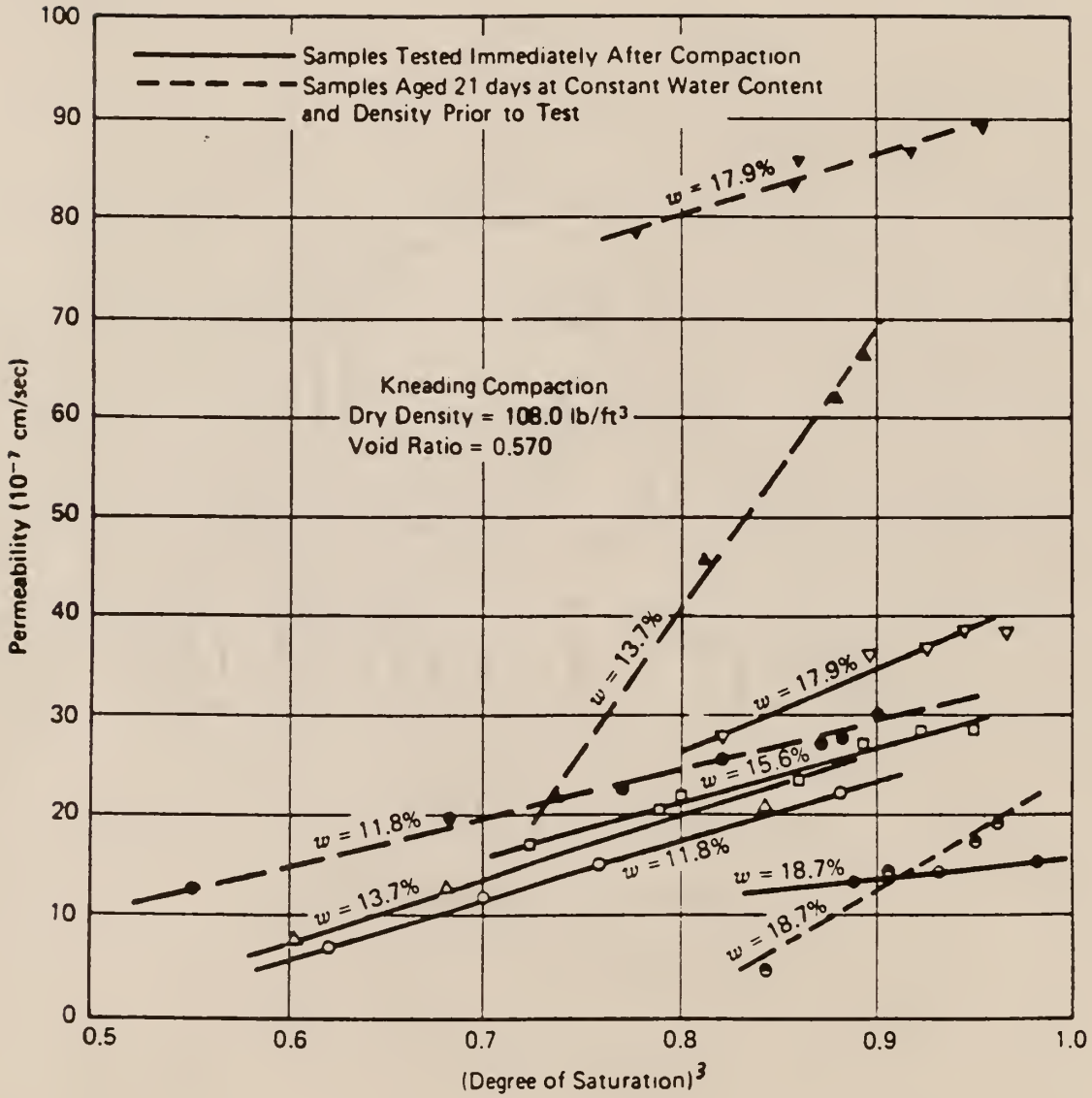


Fig. 19. Conductivity vs. Degree of Saturation vs. Aging (Mitchell 1976).

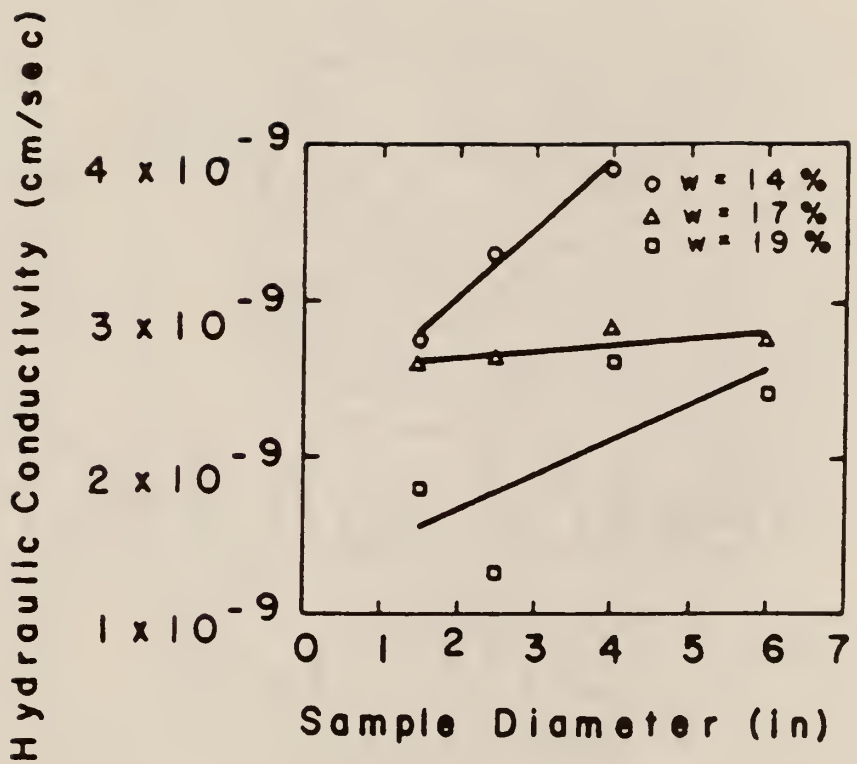


Fig. 20. Conductivity vs. Sample Diameter (Boynton and Daniel 1985).

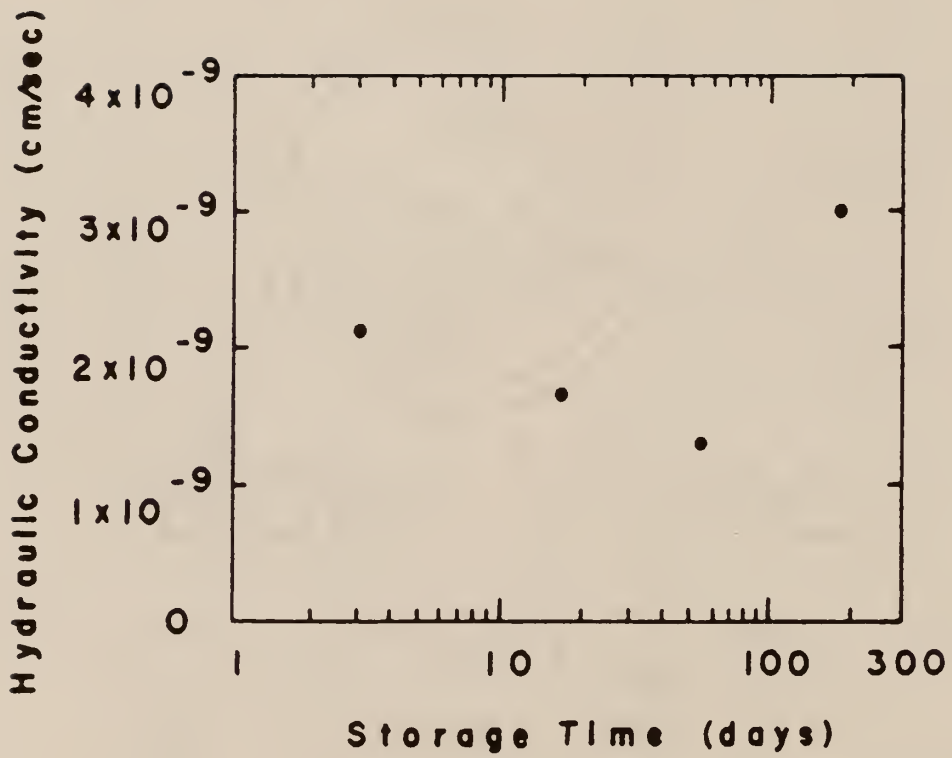


Fig. 21. Conductivity vs. Aging (Boynton and Daniel 1985).

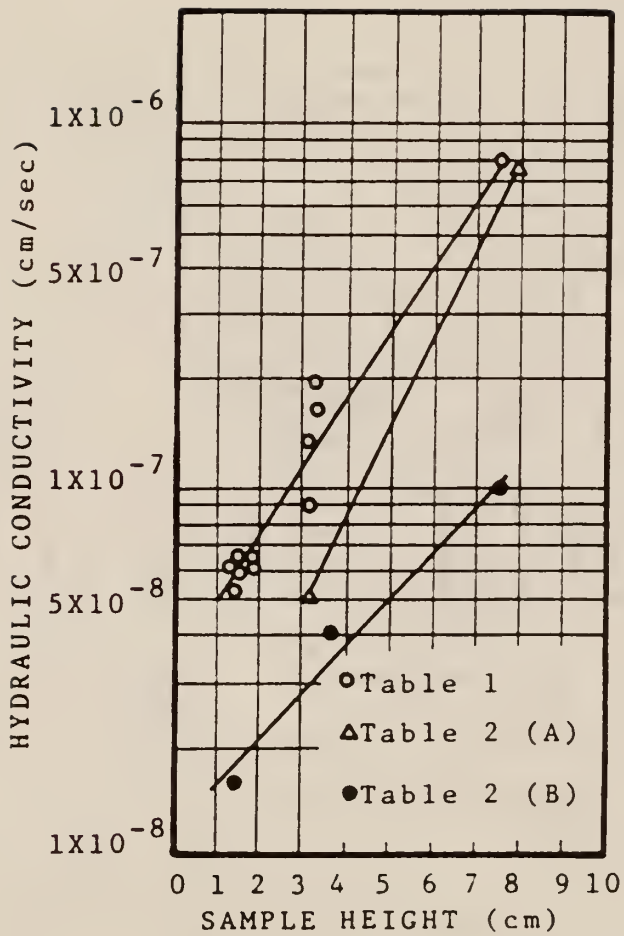


Fig. 22. Conductivity vs. Sample Height (Korfiatis et al. 1987).

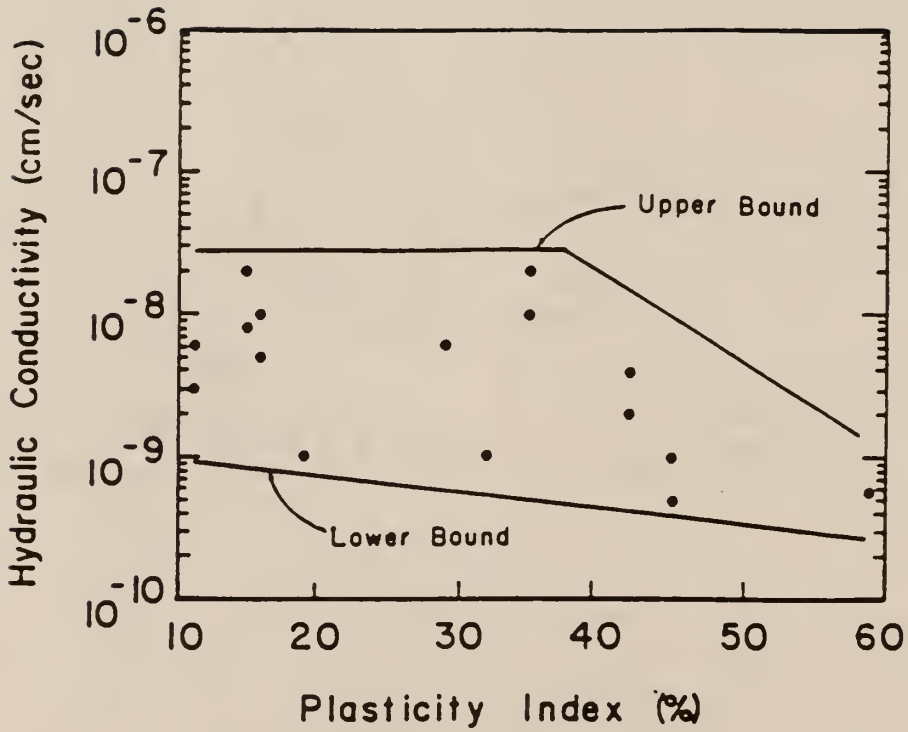


Fig. 23. Conductivity vs. Plasticity Index (Daniel 1987).

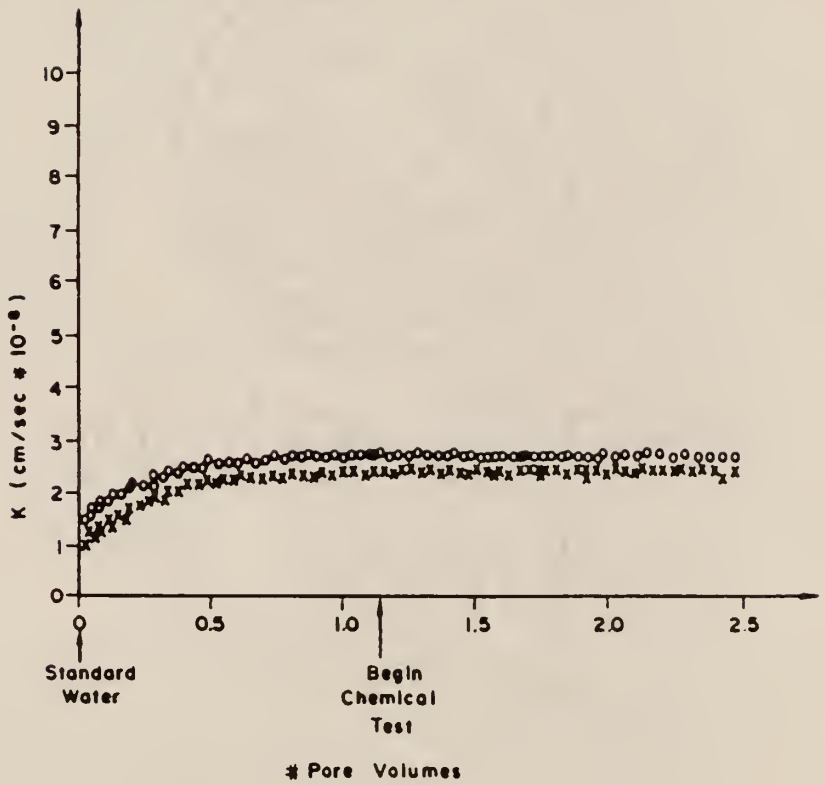
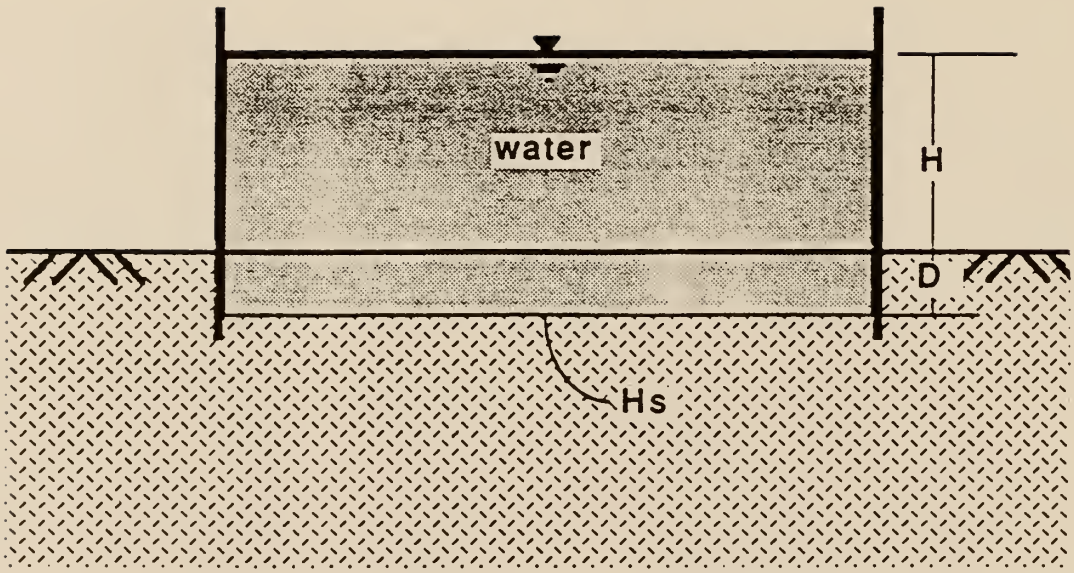


Fig. 24. Conductivity vs. Pore Volume (Peirce and Witter 1986).



$$i = \frac{H + D + Hs}{D}$$

$$Hs = ?$$

Fig. 25. Schematic of Single Ring Infiltrometer and Suction Head.

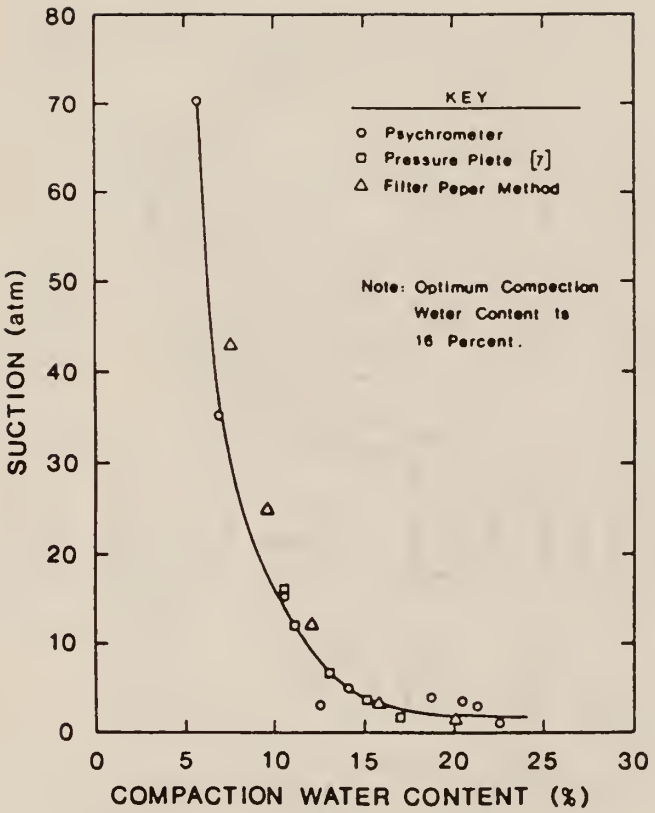


Fig. 26. Suction vs. Water Content (Daniel et al. 1979).

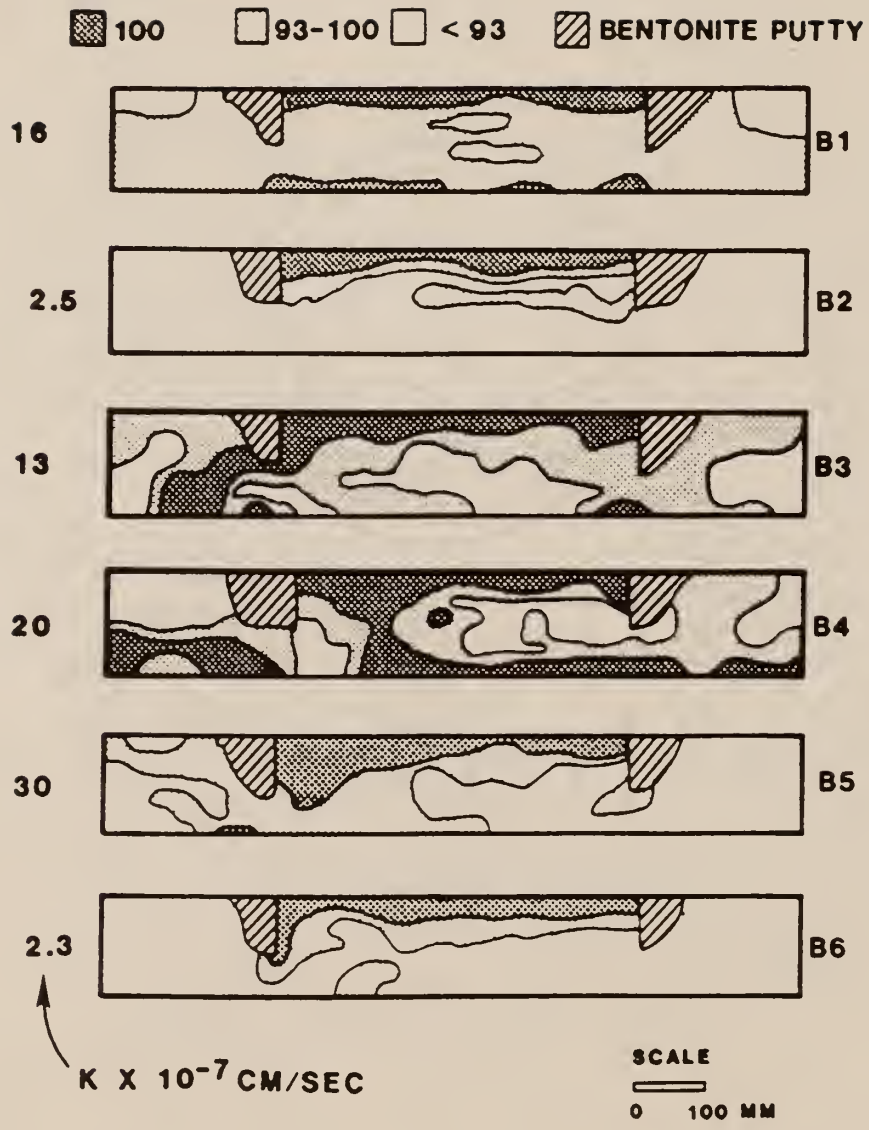


Fig. 27. Distribution of Soil Saturation after Field Infiltration Tests (Stewart and Nolan 1987).

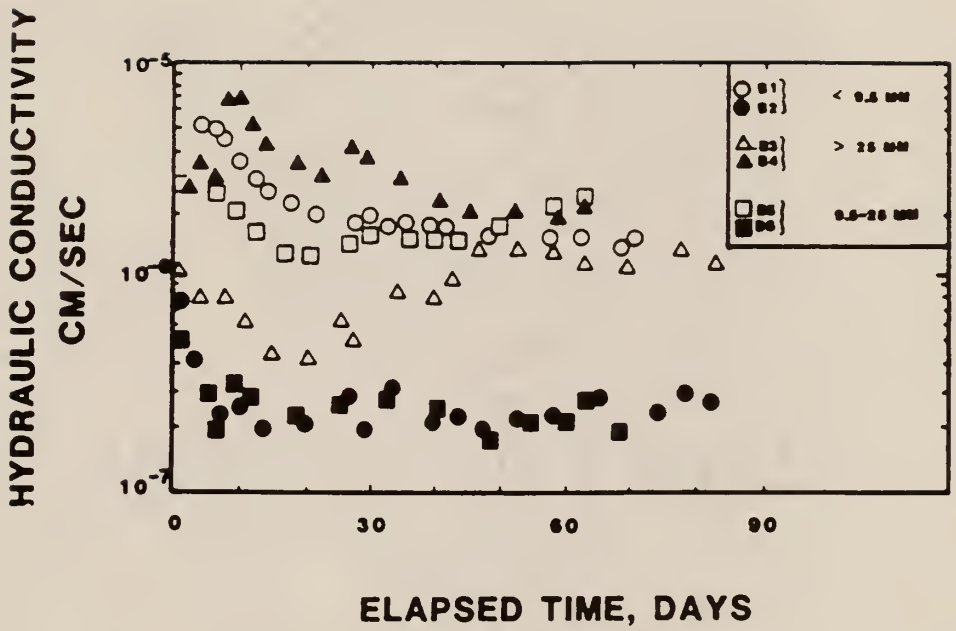


Fig. 28. Field Conductivity vs. Time (Stewart and Nolan 1987).

CHAPTER 2
BULK SAMPLING, PROPERTIES, AND SAMPLE PREPARATION

Bulk Sampling

A total of five bulk samples of the Terra-Seal Natural Premix clays were obtained from different parts of an existing stockpile at MFM surface clay mine (Lowell mine) in Ocala, Florida. The stockpile was very large and was made by excavating the natural clay deposits, mixing it, and then stockpiling it. This operation was performed in order to disrupt any existing soil stratification. The stockpile soil consisted of firm to stiff yellowish and reddish brown mottled light gray and green silty clay with a trace of fine to medium sand and a trace of fine to coarse gravel-sized limestone nodules. The stockpile also contains small to medium boulder-sized clay lumps. Approximately 1000 pounds of the clay was obtained and brought back to the University of Florida laboratory in Gainesville. In the laboratory, all of the sampled clay was placed in a large tray, mixed thoroughly, and every effort was made to insure that the nominal size of all clay lumps was not larger than 2 inches. This operation was necessary to obtain an average and homogeneous clay sample. This sample was then placed in a

tightly sealed large container, stored in a controlled environment, and was used as the project clay in all subsequent laboratory tests.

Properties of the Project Clay

The properties of the project clay consisted of index and physical, mineral, and chemical properties. Some of these properties were measured directly and some were collected from previous work performed on the same clay.

Index and Physical Properties

Index and physical properties of the clay were obtained in the laboratory by the author. These properties consisted of natural moisture content (as received moisture content), percent passing the No. 200 sieve (percent fine which represent silt and clay), Atterberg Limits, and specific gravity of the solid particles. All of these tests were performed in accordance with ASTM (1989) standard methods of testing. A summary of the results of these tests is shown in Table 4. Detailed results of these tests are shown in Appendix A. Table 4 also shows a summary of results of similar tests performed by two different professional testing laboratories on the same clay used at two different projects. Details of these test results are shown in Appendices C and D for South West Alachua and Astatula landfill projects, respectively. The table shows that the clay properties

obtained by the author are very close to the others. This suggests that the project clay can be safely assumed to be the same as that used in the two projects.

Mineral and Chemical Properties

Type and amount of minerals present in the clay were studied by Dr. James Eades and Dr. E. C. Pirkle of the Department of Geology at the University of Florida during 1988. These properties were established by a combination of hydrometer and X-ray analysis performed on a number of clay samples. They found that the clay contains 19% to 78% of fine sand, 2% to 18% silt, and 13% to 73% clay. Furthermore, the clay was found to be mainly montmorillonite with a trace of sepiolite, attapulgite, illite-waverlite, and kaolinite-weathered. Details of the mineralogical studies are included in Appendix B.

Chemical analysis of the clays was performed by Post Buckly, Schuh, and Jernigan, Inc. They perform the analysis on clay samples obtained from that used in the construction of the Astatula landfill project. Total metal tests and Toxicity Characteristics Leaching Procedure were performed on the sampled clays. As part of the total metal testing procedures, the clays were tested for arsenic, barium, cadmium, chromium, lead, mercury, selenium, silver, and sodium. It was concluded, based on these tests, that the clays meet the EPA (Environmental Protection Agency) and FDER

(Federal Department of Environmental Regulations) standards and that the clays are not hazardous to the existing surficial aquifer water quality. Detailed results of the chemical analysis are included in Appendix B.

Sample Preparation

Different sets of procedures were followed in the preparation of clay samples for laboratory and field tests. However, in each case, the preparation procedures were conducted using the recommendations suggested by well-known documented standard and nonstandard procedures.

Laboratory Samples

All samples prepared in the laboratory for conductivity tests were in accordance with ASTM (1989) standard procedures. Two particular ASTM test procedures were followed in the preparation of compacted clay samples for hydraulic conductivity tests. These were ASTM D-698-method A (standard or Proctor method of compaction for fine grained soils) and ASTM D-1557-method A (modified method of compaction for fine grained soils). Method D698A specifies that the clay should be placed in a standard mold in three equal layers, each layer subjected to 25 blows of a 5.5 pound rammer falling from 12 inches above the surface of the clay. While method D15 57A specifies that the clay is to be placed in the same standard mold in five equal layers, each layer

compacted by 25 blows of a 10-pound rammer falling from 18 inches above the surface of the clay. This means that resulting samples are subjected to higher compaction energy and, therefore, possess higher unit weight for the same molding water content than those prepared by method D698A. By the measurement of sample volume, wet weight, and moisture content, the dry unit weight and the molding water content of each sample was obtained. Detailed procedures of these methods can be found in the ASTM (1989) handbook.

The mold for the laboratory samples was made of cast acrylic plastic tubing with inside diameter of 4 inches, outside diameter of 4.5 inches, and with variable lengths. The inside diameter of the tubing is the same as that for the standard mold.

Samples used for suction measurements

Samples for the suction tests were prepared by following ASTM procedures in which the clays were air dried, passed through the No. 4 sieve, mixed with appropriate amount of tap water, cured for 48 hours, and then two samples with the same moisture content were compacted in accordance with D698A and D1557A. A rubber ring was placed on the surface of the prepared sample, a Fisher Scientific standard filter paper No. 09-790A was placed on top of the ring, and the top of sample was air-tight sealed for at least 10 days. The type of filter paper was the same as that used by McKeen (1988).

After the test termination, the moist filter paper was placed in a preweight sealable plastic bag and its weight recorded. Then, the moist filter paper was placed in a 110°C constant temperature oven for 24 hours and then in a fresh preweight sealable plastic and weight.

Samples used to study the effect of desiccation

Samples for desiccation study tests were prepared as in those for suction tests except the dried clay was mixed with about 24% moisture content (wet). This is because compacted wet soil dries more and, hence, desiccates more than compacted dry soil. Two identical 18-inch-thick compacted samples were prepared in 12 equal layers compacted in accordance with D698A. Six thermocouples were placed in one of the samples at 1.27, 3.81, 7.62, 15.24, 26.67, and 41.91 cm from the top. This was to monitor the temperature profile with time. The two samples were placed in an ultraviolet chamber with a constant temperature of 38°C. Daily readings of the temperatures of the six thermocouples were taken for 16 days. At the end of this period, moisture content profile tests were performed on the sample with thermocouples. In addition, conductivity tests were performed on the other sample. Moisture content profile tests were performed after the completion of the conductivity tests. Figure 29 shows a cross section of the adopted laboratory hydraulic conductivity setup.

Samples used to study the effect of soil thickness

Samples for the soil thickness study were prepared using the average homogeneous Terra-Seal Natural Premix soil that was discussed in the Bulk Sampling section. Four samples with thicknesses of 1.5, 4.5, 12, and 18 inches were prepared in accordance with D698A. All samples were placed in 1.5-inch layers, applying the same amount of compaction energy per layer. Then conductivity tests under a constant hydraulic gradient of 70 were performed on each sample. When the hydraulic conductivity reached a stabilized value, within 5% to 10% of the previous reading, the value was recorded and the test was terminated.

Samples used to study the effect of number of layers

Samples used to study the effect of number of layers on the predicted conductivity were prepared using the average homogeneous clays described in Bulk Sampling section. Samples with a total thickness of 1.5 and 4.6 inches were prepared in one and three layers; samples with total thickness of 12 inches were prepared in two, four, and eight layers. The total applied compaction energy per unit volume was the same for all samples and for ASTM D698A. Then, conductivity tests under a constant hydraulic gradient of 70 were performed on each sample. The hydraulic conductivity value was recorded when it reached within 5% to 10% of the previous reading.

Samples used to study other factors

The effect of increasing hydraulic gradient and time on the measured hydraulic conductivity was studied using samples from the average homogeneous clay and prepared in accordance to D698A or D1557A. When the effect of increasing hydraulic gradient is studied, the gradient is increased after a stable conductivity value is reached. When studying the effect of time, a conductivity reading was recorded after an arbitrary time.

Field Sampling

Thick wall tube sampling techniques were used to perform field infiltration tests and to obtain undisturbed field samples for laboratory conductivity and other tests. Three steel sleeves were used with a length of 6.8 and 12.8 inches, inside diameter of 4.25 inches, outside diameter of 4.5 inches, and an 0.7-inch build in cutting shoe. Acrylic plastic transparent tubing was made to slide freely inside the sleeves. The plastic tubing has exactly the same length as the housing sleeve with an inside diameter of 4 inches and an outside diameter equal to the inside diameter of the sleeve (4.25 inches). The cutting shoes were tapered by 0.41 degree (angle of taper) for one of the long sleeves and 0.96 degree for the short one. This will allow slightly more clay to flow inside the plastic tubing upon pushing into the clay and, hence, facilitate a better bond or sealing against the

inner surface of the plastic tubing. This eliminated and/or minimized water flow along the interface between the clay and the plastic tubing. The cutting shoe of the other long sleeve was not tapered. This was made to establish the effect of tapering on the clay unit weight inside the plastic tubing. A steel cap was made to fit on top of all three sleeves. This cap allowed retrievement of the sleeve, plastic tubing, and the clay after it has been pushed into the clay. Figure 30 is a cross section of the above-described apparatus with all the related dimensions.

Field sample disturbances

Field sampling of in-situ soils will always disturb the in-situ soils. There are a number of factors that affect the type and amount of disturbance of the soils during field sampling, and they are different for different types of soils and soil conditions. However, the following is a summary of the main types and amounts of disturbances that are associated with the performed field infiltration and sampling. The amount of disturbances was obtained or deduced from the comprehensive study conducted by Hvorslev (1962) on sample disturbances.

Forces and deformations during sample driving

During field sampling, the combined steel sleeve and plastic tube (sampler) were driven into the soil. Sample

driving is a process where a tube is forced into the soil without any rotation or chopping action and without removing the soil displaced by the walls of the sampler. During driving, the soil is subjected to some forces and deformations.

Forces during the driving. During driving there are forces developed outside and inside the sampler. These forces are acting on the inside and outside of the sampler and are due to side friction, side adhesion, weight of the soil, and lateral forces. In the case of this study, outside forces are not of interest and inside forces are minimal. This can be explained as follows: because the length of driving is short (6 to 12 inches), the internal surface of the sampler is smooth, and because of the plastic nature of the clay, a very low developed angle of internal friction over short distance results; side adhesion is low due to the short length of driving and the clay being placed in a relatively dry condition, resulting in low moisture content which will lead to low adhesion; inside and outside lateral forces together with the soil weights are very small due to the fact that the sampled clay is located at the surface (no large overburden) and the shallow nature of driving distances.

Entrance of excess soil. As the sampler advances, part of the soil under the annular area (due to the wall thickness of the sleeve) is displaced by the walls of the sampler and

pushed aside, and the other part will enter inside the sampler causing some deformation to the sampled soil. This deformation is in the form of slight soil densification and pores closing in the peripheral region of the soil sample. The amount of soil entered will depend on sampler dimensions, angle of taper of the cutting shoe, method of driving, driving distance, and soil conditions. The first three factors are dealt with below. Disturbances due to the last two factors are minimal because of the very shallow driving distance, surficial nature of the sampled soil, and stiff consistency of the in-situ soil.

Dimensions of the drive sampler

The principal dimensions that are influencing soil disturbances are area ratio, diameter of the sample, and length of the sample.

Kerf or area ratio. The penetration resistance of the sampler, the possibility of entrance of excess soil, and danger of disturbance of the sample all increase with increasing area ratio. Area ratio is approximately equal to the ratio between the volume of displaced soil and the volume of the sample or can be expressed in terms of diameters as follows:

$$A_r = [(D_o^2) - (D_i^2)] / (D_i^2) \qquad (28)$$

where

A_r = Kerf or Area ratio,

D_o = outside diameter of the steel sleeve (4.5 inches), and

D_i = inside diameter of the plastic tube (4 inches).

Therefore, A_r = 26.6% for the sampler used in this project.

Hvorslev found that soil disturbance is negligible for an area ratio of 10 to 14%, very low for an area ratio up to 45%, and high for an area ratio equal to or greater than 79%. Based on the area ratio used in this study (26.6%), soil disturbance is very low. He also found that disturbance due to area ratio is almost negligible for a driving depth of less than 15 inches and greater than 35 inches, and the entrance of excess soil due to area ratio is highest for soft plastic clay and small for stiff cohesive soil.

Diameter of the sample. Hvorslev did not study the influence of sample diameter on disturbance. But he suggested that disturbance of the soil is highest close to the cylindrical surface of the plastic tube, and this disturbance decreases with increasing diameter and becomes negligible for a sample diameter equal to 3 inches. Since the sample diameter used in this study is 4 inches, the disturbance due to sample diameter is minimal.

Length of sample. Hvorslev found that a safe length of sample that results in minimal soil disturbance depends on

many factors but can be best expressed in terms of inside diameter of the plastic tube.

$$L_s = 10 \text{ to } 20 * D_i \tag{29}$$

where

L_s = length of sample.

He also found that the safest length is 2 to 3 times the inside diameter of the plastic tube (for a 2- to 3-inch diameter sampler). The maximum length of the sampler used in this project was 12.8 inches while the inside diameter of the plastic tube used is 4 inches. Therefore, the used sampler is within the safest range, and, hence, soil disturbances due to the length of sampler is minimal.

Cutting shoe. The disturbance of soil due to the taper angle of the cutting shoe was studied by Hvorslev (1962) using the M.I.T and Mohr samplers with an area ratio of 44% and angle of taper ranging from 13 to 20 degrees. It was found that no excess soil was entered for a sampler with a taper angle of 13 to 14 degrees. The cutting shoe used in this study was a maximum of 0.96 degree. Therefore, this effect can be assumed to be very negligible.

Driving method

Hvorslev has suggested that a fairly uniform and uninterrupted advance at 0.5 to 1.0 foot per second produces less disturbed samples than either hammering or slow jacking. In this study, the sampler was pushed, for the first two infiltration tests, by jacking at a rate of 1.0 foot per 10 minutes. All subsequent tests and samplings were pushed in by using a backhoe at a rate of 1.0 foot per minute.

TABLE 4. Comparison of Range of Index and Physical Properties of the Project Clay

Project--Tested by	W%	% Fine	Atterberg Limits LL	PL	PI	F. Class	Soil Classification USCS	G_s
Project Clay by Author	15-22	38-70	45-54	17-18	18-36	CL-CH	SC CL-CH	2.48-2.64
SW Alachua by Ardaman & Assoc.	17-30	27-50	35-70	14-20	21-50	CL-CH	SC CL-CH	--
Astatula by Jammal & Assoc.	15-27	33-54	44-78	16-27	28-51	CL-CH	SC CL-CH	--

Key: W = Natural moisture content
% Fine = % Passing No. 200 sieve
LL = % Liquid limit
PL = % Plastic limit
PI = % Plastic index
F. Class = Fine classification (USCS)
G_s = Specific gravity

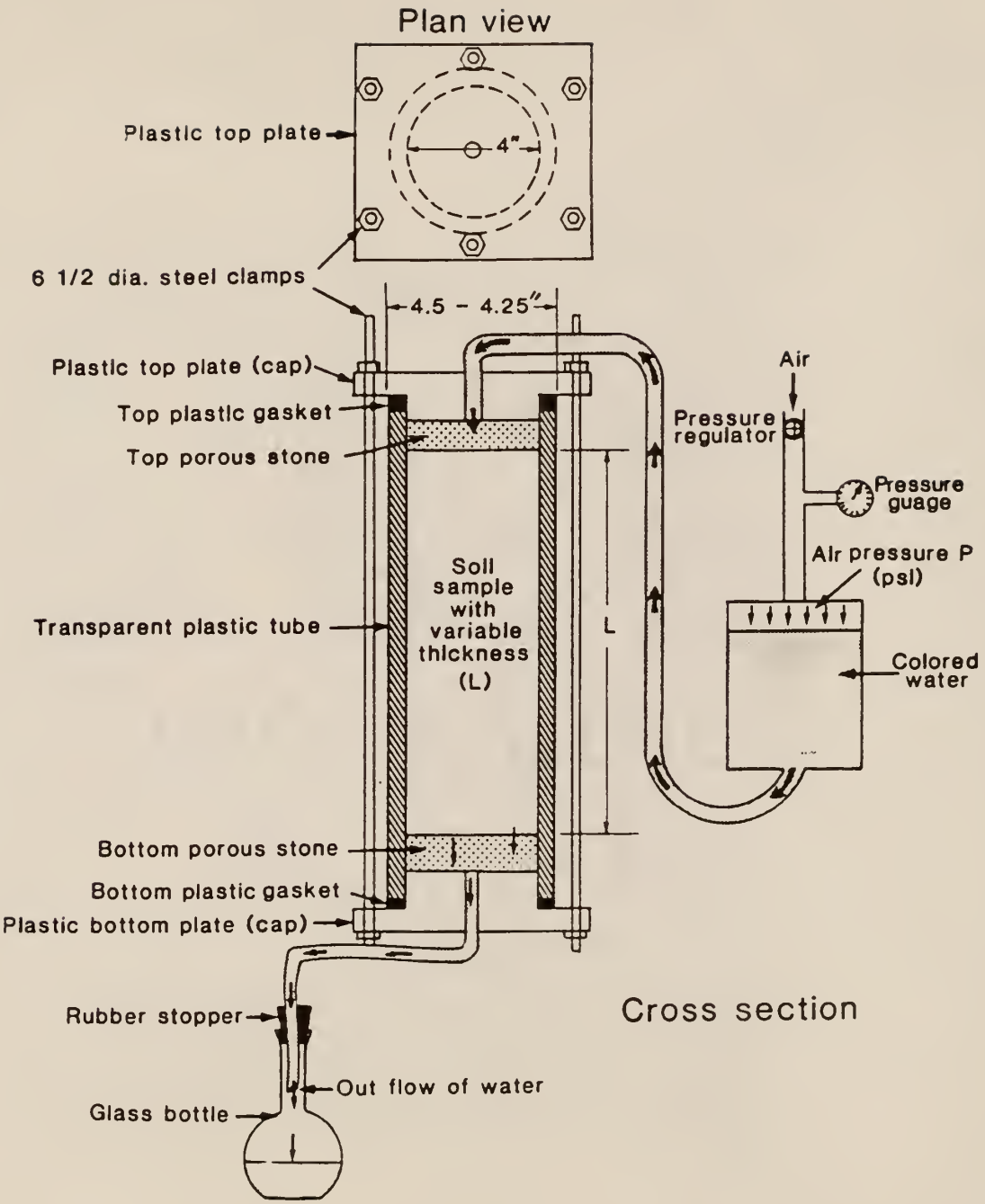


Fig. 29. Cross Section of the Laboratory Rigid Wall Permeameter.

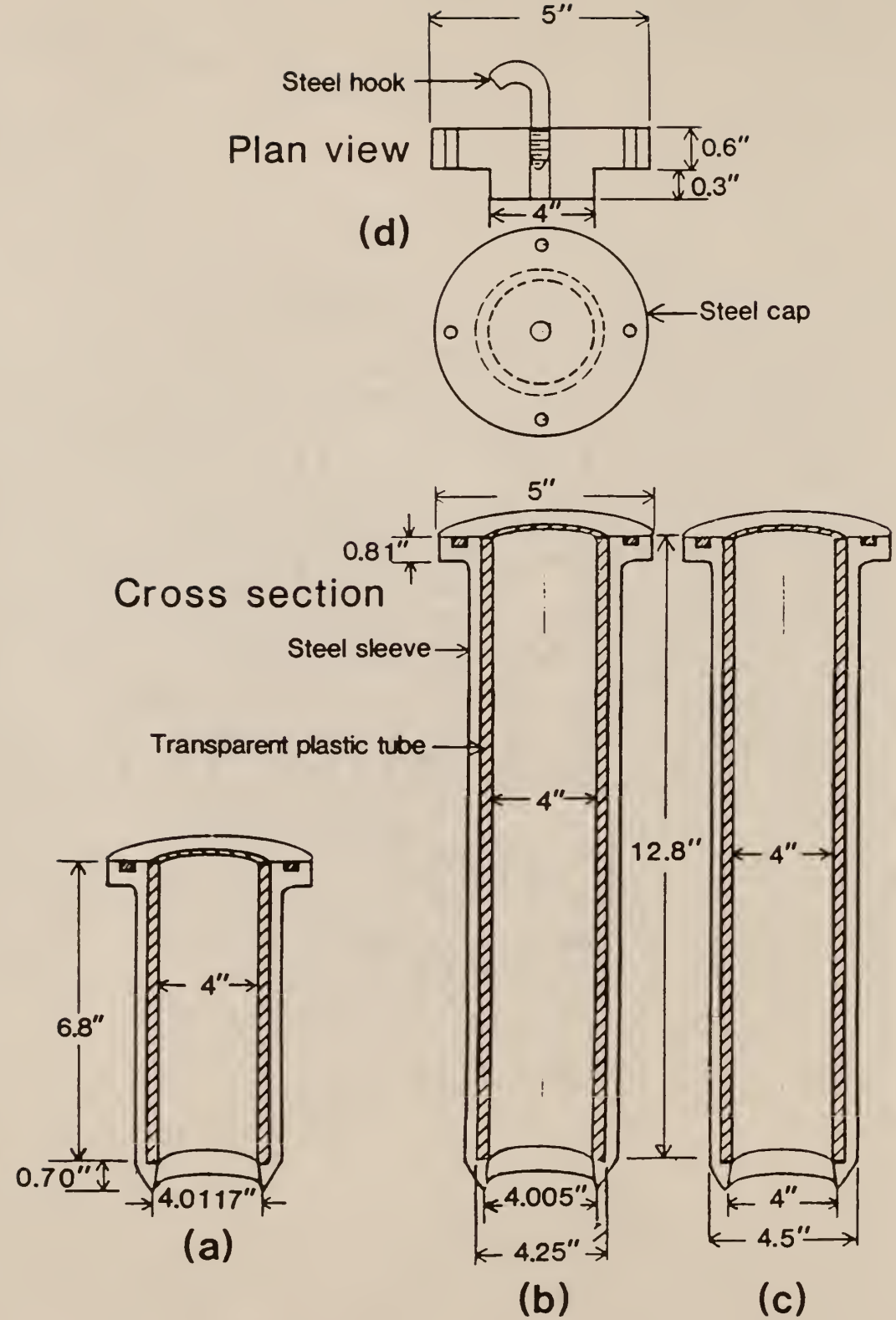


Fig. 30. Cross Section of the Steel Sleeves Used in Field Infiltration Test and Undisturbed Sampling.

CHAPTER 3
LABORATORY TESTS, RESULTS, AND DISCUSSIONS

Laboratory Hydraulic Conductivity Tests

Hydraulic conductivity tests were performed in the laboratory on the prepared compacted clay samples as discussed in Chapter 2. The permeameter utilized was the rigid wall permeameter. Saturated porous stones were placed on the top and bottom of the prepared samples. The saturation was achieved by boiling the porous stones until no air bubbles were observed (about 2-minute periods). The samples were then placed between two acrylic plastic plates that were tied together by three to six 0.5-inch diameter adjustable steel clamps. Plastic gaskets were placed between the top and bottom plastic plates and the samples. This was done in order to prevent leakage especially at higher hydraulic gradients. The thicknesses of the plastic plates, porous stones, and the plastic gasket were designed such that no or very minimal soil swelling was allowed.

Ordinary tap water was fed to the top of the sample through a 0.25-inch high pressure plastic tube. This plastic tube was connected to a 4-inch diameter and 8-inch high acrylic plastic water reservoir. Incoming air was

pressurized using an air pressure regulator and then applied to the upper water surface in the reservoir. The amount of applied air pressure in psi (pounds per square inch) was monitored and recorded. The amount of required air pressure (P in psi) was calculated based on the desired hydraulic head (H_o in cm) which was a function of hydraulic gradient (i in cm/cm) and the length of the tested sample (L in cm).

$$i = H_o/L = (P * 1006.5)/L \tag{30}$$

Therefore, by knowing L and i, H_o and P can be calculated. The maximum allowable air pressure that could be applied was limited by the strength of the water reservoir and was about 100 psi.

After the application of air pressure the out flow of water was collected from the bottom of the sample by a glass bottle. This bottle was air tight and sealed by a rubber stopper to prevent evaporation of the outflow. After a period of time, the amount of water outflowed was measured by weighting techniques using an electronic scale accurate to 0.01 gram (g).

The water in the reservoir was colored first using Rhodamine and, later, by ordinary food coloring. This was done in order to visually monitor the movement of water through the interface of the soil sample and the plastic tube and to observe the path or paths of moving water through the

soil sample after the termination of the hydraulic conductivity test. The types of coloring were chosen based on their inert reactions with the used soil. This is a fact well established by a number of University of Florida and outside researchers. Figure 29 show a cross section of the laboratory test setup that was used together with all the parts and dimensions discussed above.

Soil Suction and Saturation vs. Density vs. Moisture Content

General

The relationship between soil suction, dry unit weight, and moisture content was studied using 14 soil samples 4 inches in diameter and 4.6 inches high prepared in accordance with the procedures discussed in Chapter 2 (samples used for suction measurements). These samples were prepared in pairs with a moisture content ranging from 3 to 24.2%. In each pair, the first sample was compacted using ASTM D698A (standard method of light compaction), and the other sample was compacted using ASTM D1557A (modified method of heavy compaction). In addition, another pair of samples was prepared using the actual Terra-Seal Natural Premix® at the field moisture content. Filter paper suction tests were performed on each sample as discussed in Chapter 2. All operations involving filter paper weighting and handling were done using a scale sensitive to 0.00001 g and tweezers, respectively.

Dry Unit Weight vs. Moisture Content

The calculated dry unit weight with moisture content for the samples compacted in accordance to ASTM D698A and D1557A were plotted, and an average curve was drawn for each type of compaction. The resulting curves are shown in Fig. 31. These curves are of a standard shape and very similar to those obtained using the same clays, shown in Appendices C and D. The curves show that at a moisture content of about 3% the dry unit weight is lowest. This is due to the soil structure being highly flocculated with a high degree of porosity. As the moisture content increases, so does the dry unit weights up to maximum values. As the moisture content increases, leading to the lubrication of soil particles which facilitate sliding over one another, it results in breaking up of flocculations and in decreasing degree of porosity leading to higher unit weights. For the same moisture content, the dry unit weight of samples prepared in accordance with D1557A is consistently higher than those for samples prepared in accordance with D698A. This is because samples prepared in accordance with D1557A are subjected to higher applied compaction energy than those prepared in accordance to D698A. This higher energy will force soil particles together at low moisture content resulting in higher unit weights. The maximum dry unit weight and optimum moisture content for samples compacted in accordance with D698A was 103 pcf (pounds per cubic foot) and 19%,

respectively, and for those prepared in accordance with D1557A was 114 pcf and 15%,respectively. At the point of maximum unit weight and optimum moisture content, the soil structure is semidispersed (dispersed with little flocculations), resulting in the lowest degree of porosity. As the moisture content increases above the optimum values, the excess moisture will force soil particles apart, resulting in a dispersed soil structure, higher degree of porosity, and, hence, lower unit weights.

Degree of Saturation

Degree of saturation (S) is the ratio of volume of water to volume of voids in the sample and expressed as a percentage. The degree of saturation (S) for each sample was calculated using the principle of 3-phase diagram and an average solid specific gravity (G_s) of 2.55 (Table 4). The calculated values are superimposed on the dry unit weight-moisture content curves as shown in Fig. 31. Curves were drawn through the average values resulting in the equal degree of saturation lines shown in Fig. 31. The average values of the degree of saturation are indicated on the respective curves. The shape of these lines is not exact but rather approximate and are drawn on the basis that all lines should be parallel to the 100% line. The main reason for the observed scatter of saturation is due to the assumption that all soil solids have a single and uniform specific gravity.

However, a more accurate average shape could be determined if more samples were tested resulting in minimizing the effect of nonuniformity of specific gravity on the saturation data. Saturation lines show that at the lowest moisture content (3%) the degree of saturation is lowest and is equal to 13%. As the moisture content and unit weights increase, the amount of water increases and the volume of voids decreases, resulting in a higher degree of saturation. At the maximum unit weights and optimum moisture content, the degree of saturation is approximately 90%. At a moisture content above the optimum value, although the volume of voids increases resulting in lower unit weights, the amount of increase in the volume of water is higher relative to the increase in the volume of voids leading to a higher degree of saturation.

Suction

Soil suctions were calculated using the procedures suggested by McKeen (1988) which are outlined earlier in Chapter 1. The actual equations that were followed in the calculations were as follows.

$$W_f = (W_w - W_d)/W_d \quad (31)$$

where

W_f = filter paper water content (fraction),

W_w = moist weight of the filter paper (g.), and

W_d = oven dry weight of the filter paper (g.).

Then, the suction heads (h) in cm of water, expressed in terms of P^F, were calculated using the average equation suggested by McKeen (1988).

$$h = 6.00 - 7.00 * W_f \tag{32}$$

This equation was developed by McKeen by combining all the curves shown in Fig. 15 into an upper and lower bound zone and drawing a line through the arithmetic averages of this zone. This is shown in Fig. 32 (McKeen 1988). Like the values of the degree of saturation, the calculated soil suction values were plotted on the same dry unit weight-moisture content curves, as shown in Fig. 33. Then, straight lines are drawn through the suction values that do not vary by more than 0.5 to 1.0 P and are located within close regions with similar unit weight and moisture content. Figure 33 shows these lines together with average values of the suction in P^F indicated on the respective lines.

All previous works have assumed and studied the soil suction as a function of moisture content only and found that for a given type of soil, as the moisture content increases,

the suction decreases. Typical of such a result is shown by Fig. 26 in Chapter 1. Furthermore, McKeen (1988) has found that the soil suction ranges from P^F of 6 to 3.5 for driest and wettest soil, respectively, while Fig. 26 (Daniel 1979) shows that this range is between P^F 5 to 3. However, for a given type of soil the amount of suction depends not only on the moisture content but also on the unit weight of the soil. This is explained by the equation for the capillary rise in a tube which is usually applied to estimate the capillary head rise in soils (Lambe and Whitman 1979, Mitchell 1976, Taylor 1948, Sing 1967).

$$h_c = (2\ T_s * \cos \alpha)/(R * \gamma_w) \qquad (33)$$

where

h_c = height of rise in a capillary tube,

T_s = surface tension of liquid,

α = contact angle made between the liquid and the tube,

R = radius of the tube (or radius of pore in soil), and

γ = unit weight of water.

Equation 33 suggests that the suction head is inversely proportional to the diameter of the pore spaces in the soil. When the soil samples have fictitiously very high unit weights, there will be no pores (R = 0), and, consequently, there will be no suction. When the unit weight of soil

samples decrease by a very small amount, there will be very few pores with very small diameter (R = very small number), and, consequently, the amount of suction is very high. As the unit weight of the soil sample becomes very low, this means a high degree of porosity (higher number and larger sizes of pore spaces) and a larger value of R, which will lead to very low suction, where the degree of porosity is equal to the ratio between the volume of pores to the total volume of the soil sample.

The results of applying the above analysis to the suction tests are shown in Fig. 33. Close observation of the plotted suction results (actual points) reveals two important points:

1. For the same moisture content, soil samples with higher unit weights consistently display higher suction values. This fact is a further reinforcement of the relationship between pore spaces and suction as described by equation 33. In other words, the higher the unit weight, the lower the degree of porosity, the lower the diameter of the pore spaces, and, hence, higher suction. It must be understood that the reduction in degree of porosity of clay soils, upon higher degree of compaction, will be mainly due to the reduction in the amount and size of pore spaces between clods and, to some degree, between cluster of clay particles. The micro pore spaces between individual clay particles will remain practically unchanged for moisture

contents below the optimum values. This means that clays with this range of moisture content will always have some suction regardless of its unit weight. At moisture content above the optimum values, the double layer around each clay particle will be fully developed, and individual clay particles start repulsing each other, and, hence, the pore spaces between those individual clay particles will be available to participate in the influencing of the suction value of the soils. This means that even when the soil is perceived to be fully saturated, it still will have some suction. This phenomenon is reinforced by the already established, by many researchers, relationship between suction and moisture content alone. Figure 26 (Chapter 1) is a typical representation of this fact.

2. As the moisture content increases, some of the pore spaces between the clods and some clusters of clay particles will be occupied by the water, therefore, reducing the amount of suction. The water does not fill these pore spaces completely or uniformly. This will lead to artificially reduced suction (apparent suction) and will lead to nonlinearity and data scattering of the suction values. With an increase in moisture content, the pore spaces between particles tend to be nearly uniform, allowing the excess water to occupy the pore spaces relatively quickly and uniformly.

The analysis in the above two points represents the micro-scale explanations of the general trends displayed by the suction lines, shown in Fig. 33, and the corresponding average suction values indicated on the lines. A more exact shape of these lines or curves can be obtained once more samples are tested and, hence, more suction values can be plotted and analyzed.

In reality, the suction lines should complement the saturation lines shown in Fig. 31. This is because both soil suctions and saturation values are plotted on the same dry unit weight-moisture content curves. It can be seen in both Figs. 31 and 33 that there is a general trend of decreasing suction values as the degree of saturation increases and that both types of lines have approximately the same shape. However, there is a slight nonlinearity between suction lines and saturation lines. This nonlinearity can be explained by the fact that soil suction is a function of the unit weight and moisture content of the soil and type and amount of fines (clay particles), while soil saturation is a function of unit weight and moisture content and the specific gravity (G_s) of soil solid. The value of specific gravity is not uniform for a given soil sample, nor it is the same for all soil samples due to the fact that each soil sample possesses different amounts and types of minerals. An average value of specific gravity of 2.55 was used in calculating all of the saturation lines shown in Fig. 31.

Hydraulic Conductivity vs. Sample Thickness

General

The effect of soil sample thickness on the hydraulic conductivity was studied using four samples with thicknesses of 1.5, 4.5, 12, and 18 inches. The preparation of these samples was discussed in Chapter 2. However, the samples were prepared using the average homogeneous Terra-Seal Natural Premix clays, placed in 1.5-inch-thick layers, each layer compacted by 25 blows of a 5.5-pound rammer dropping from a height of 12 inches (ASTM D 698A). The clays were placed such that the maximum clod size was about 0.5 to 0.75 inch. Furthermore, the interfaces between layers were scarified prior to the placement of the net layer. All prepared samples were tested in the rigid wall permeameter cell under a hydraulic gradient of 70, and their hydraulic conductivities were recorded after reaching a stabilized value (within 5 to 10% of the next reading). The period between two consecutive readings was kept to a minimum of 24 hours. The hydraulic gradient value was chosen such that the maximum allowable air pressure was not exceeded. In addition, a hydraulic gradient of 70 will not result in a large immediate consolidation of the soil samples.

Dry Unit Weight and Porosity

The dry unit weight and degree of porosity of each sample were calculated. These calculated values are plotted

against sample thicknesses and are shown in Fig. 34. Figure 34 also shows that the method that was followed in preparing the samples (D698) and that the clay used was the natural clay as indicated by the letter "N" after D698.

The dry unit weight of the samples ranged from 95 to 105 pcf. This means a maximum difference of 10%. This difference is tolerable in the presence of various errors that are associated with all laboratory testing. However, there is a general trend of decreasing unit weight as the thickness of the sample increases. This can be explained by the increasing possibility of the existence of pockets of sandy soils and uncompacted soils, resulting in a relatively large isolated void space. As discussed before, as the unit weight decreases, the degree of porosity increases. In fact, the unit weight and porosity are interrelated and interdependent. The degree of porosity increases from 40 to 43.7% as sample thickness increases from 1.5 to 18 inches. This means the porosity values varied by about 6%. The porosity (N) line shows the same but opposite trend to that of the unit weight. The principle error in calculating the porosity is the assumption of one and a uniform value of specific gravity of 2.55.

Saturation and Hydraulic Conductivity

The degree of saturation values ranged from 86 to 100%, while the values of hydraulic conductivity ranged from about

0.5 to 2.3 * 10^{-8} cm/s. The degree of saturation was calculated using the moisture content. A number of observations and deductions can be made when analyzing the saturation and conductivity curves.

As it can be seen in Fig. 34, the saturation and conductivity of the 12-inch-thick sample are much different relative to those for other samples. If the saturation and conductivity of the 12-inch sample were not taken into account, then, the saturation and conductivity range of values would be 91 to 100% and 2.17 to 2.26 * 10^{-8} cm/s, respectively. In this case, the degree of saturation increases with increasing sample thickness. This is due to the fact that as the sample thickness increases, the distribution of the applied compaction energy is such that it is highest for the bottom layer (first layer) and lowest for the top layer (last layer). This will cause the soil moisture to be worked more uniformly throughout the soil sample or the moisture will be forced to migrate from highly moist zones to less moist zones (upward and downward). This will make the measured moisture content higher than usual, leading to higher degree of saturation.

The conductivity values, on the other hand, varied only by 4%, which is negligible. Therefore, in this case the deduced conclusion is that hydraulic conductivity is independent of the sample thickness and, for all practical purposes, can be taken as constant. This conclusion is very

plausible since the conductivity of three out of the four different thicknesses displayed a constant value. However, there are a number of factors that cause the conductivity value to be changed with increasing thickness of the soil sample. These factors are interrelated, and some of them will cancel the influence of the other. These factors are as follows:

1. As the sample thickness increases, the cumulative compaction energy will be highest and lowest for the lower and upper parts of the soil, respectively. This will result in higher and lower unit weight in the lower and upper parts of the soil, respectively. This, in turn, leads to the conductivity value being the same as that for the thin sample in the upper part and lower value in the lower parts. This will mean lower average conductivity. But, as discussed previously, as the sample thickness increases, the possibility of the existence of low compacted zones and zones with high sand content increases. This will increase the conductivity value, hence cancelling the effect of the changing unit weight distribution.

2. As discussed above, as the sample thickness increases, the distribution of moisture content will change and will be more uniform toward the bottom and more erratic toward the top. This will result in higher conductivity at the upper part and lower conductivity toward the bottom of the sample. The net result is lower conductivity of the soil

sample, especially if the original moisture content is near the optimum value. This is because at the end of compaction the moisture content at the bottom will actually be higher than the original one due to compaction, and this will lead to lower conductivity. This is supported by the relationship shown in Figs. 16 and 17 in Chapter 1.

3. As the thickness of the soil sample increases, so does the number of placed layers. This will mean a higher number of interfaces and a higher number of relatively thin, homogeneous high unit weight films or layers. These films or layers develop due to compaction and are located within 2 inches from the top surface of each layer. The hydraulic conductivities of these films or thin layer films are very low. Some of these low conductivities will be cancelled due to the higher conductivity values at the interfaces. The net result is lower hydraulic conductivity of the tested sample.

Figure 34 also shows that the shape and trend of the conductivity curve is the same as that for the degree of saturation. This shows that the initial degree of saturation has an important influence on the conductivity. This is because a lower degree of saturation means higher sample suction and potential for retaining water upon testing. This will mean a higher infiltration rate and an increase in the trapped air in the voids upon testing, which will result in the blocking of some of the pores. This leads to lower conductivity and is the main reason why the 12-inch-thick

sample displayed lower conductivity than the other sample. This low saturation might be due to nonuniformity in soil selection prior to compaction. The low conductivity can also be explained by the possibility that the prepared sample possesses a lower degree of effective porosity (which is the volume of the connected pore spaces only and it is lower than the porosity of the sample).

Hydraulic Conductivity vs. Number of Layers

General

The effect of the number of layers used to make up the soil sample on the hydraulic conductivity was studied using three samples with total thicknesses of 1.5, 4.6, and 12 inches. An outline of the method of sample preparation used to study the effect of this factor is stated earlier in Chapter 2. However, samples with thicknesses of 1.5 and 4.6 inches were prepared in one and three equally thick layers. The 12-inch-thick samples were prepared in eight, four, and two equally thick layers. For a given sample thickness, the total applied compaction energy was the same regardless of the number of layers. As an example, the total applied compaction energy for the 12-inch-thick sample prepared in eight layers was the same as for that prepared in four layers and for that prepared in two layers. ASTM D 698A was followed in the preparation of all samples. Each sample was

tested for hydraulic conductivity under the same hydraulic gradient of 70.

Dry Unit Weight, Porosity, and Saturation

The dry unit weight, degree of porosity, and the degree of saturation were calculated for all prepared samples. These calculated values were plotted and shown in Figs. 35, 36, and 37 for samples with thicknesses of 1.5, 4.6, and 12 inches, respectively. By close inspection of the curves for the dry unit weight, porosity, and saturation in the three figures, a number of observations and deductions can be made.

Samples placed in one layer have a higher degree of porosity and, therefore, lower unit weight than samples placed in multiple numbers of layers. This can be explained by the fact that with samples placed in one single layer, the applied compaction energy did not transfer well and uniformly through the full thickness of the samples, and, consequently, the lower parts of the samples did not compact well, resulting in a higher degree of porosity and, hence, lower dry unit weight. In fact, only the top 0.5 to 2 inches were observed to take most of the compaction energy in single-layer samples. For the 1.5- and 4.6-inch-thick samples placed in one single layer, the degree of porosity is about 8% higher, and the dry unit weight is about 4 to 6% lower than those for sample placed in three layers. This trend shown is by the respective curve in Figs. 35 and 36.

In the case of the 12-inch-thick sample that was placed in two layers, the unit weight was about 1 and 5% lower than from sample placed in four and eight layers, respectively, while the respective porosity was about 1 and 6% higher than those for the sample placed in four and eight layers, respectively. This means that placing the sample in two or four layers did not change the unit weight and porosity by any significant amount. This can be explained by the same reasoning as that given in the previous paragraph. In addition, in the two-layer sample, each layer was observed to have three equally thick distinct zones. An upper well compacted zone, a middle less compacted zone, and a lower least compacted zone. The compaction states of these zones did not change after the placement of the next layer. This trend was less visible in samples placed in four and eight layers. This shows that the maximum effective travel distance of the applied compaction energy (for D698A) is about 3 inches. Figure 36 shows the curves for the unit weight and porosity vs. number of layers for the 12-inch-thick samples.

The three figures also show that the effect of the number of layers on the unit weight and porosity of the samples increases for thinner samples for the same total compaction energy. This means that for thick samples the resulting unit weight and porosity do not change

significantly by preparing them in a larger number of layers. This is very important in landfill construction.

Also from the three figures the following useful rule-of-thumb can be deduced. For every 1% decrease in unit weight there is a maximum 2% increase in the degree of porosity. This is very important since the hydraulic conductivity is always related to the porosity of the soil.

The three figures also show the relationship between the degree of saturation and the number of layers. With the exception of the 1.5-inch three-layer sample (Fig. 35), the degree of saturation increases with an increasing number of layers. This can be explained as in the section, Hydraulic Conductivity vs. Sample Thickness, which is that the more layers, the more the compaction energy is uniformly distributed through the sample. This will facilitate more moisture migration and, hence, higher measured moisture content. This higher moisture content is divided by lower volume of pore spaces, as the porosity decreases for higher number of layers, resulting in a higher degree of saturation. The degree of saturation for the three-layer 1.5-inch sample is suspected to be erratic due to sample selection and, to large extent, due to the fact that the average moisture content was used in the calculation of the saturation.

Hydraulic Conductivity

As can be seen in the three figures, as the number of layers increases, the sample hydraulic conductivity decreases. This can be explained by the fact that as the number of layers increases, the porosity decreases leading to higher unit weights, resulting in lower conductivity. This is evident in the 1.5- and 4.6-inch samples prepared in three layers. The increase in conductivity was 93 and 56%, respectively, over those samples prepared in single layer. The same trend is seen in Fig. 37 for 12-inch-thick samples. One interesting observation that can be made from Fig. 37 is that for the sample prepared in two layers, the conductivity is 77% higher than that for the four-layer sample even though the increase in porosity was less than 1%. This can be explained by the fact that most of this increase in porosity was added to the value of the effective porosity. Another interesting observation is that although the 12-inch-thick eight-layer samples have higher porosity than 1.5- and 4.6-inch-thick three layer samples, their conductivity was relatively much lower than those for the others. This is due to the fact that the 12-inch-thick eight-layer samples have a greater thickness and, most importantly, low effective porosity resulting from the relatively higher unit weight of the lower layers than the top ones.

Hydraulic Conductivity vs. Hydraulic Gradient

General

The effect of the hydraulic gradient on the conductivity was studied using two 4.6-inch-thick samples prepared in accordance with ASTM D698A, as discussed in Chapter 1. One sample was prepared in one layer, and the other prepared in three equal layers. The applied hydraulic gradient was varied from 70 to 550. At each gradient and every 24 hours outflow of water was measured and the conductivity was calculated. This was repeated until a stabilized conductivity value was reached; then, the gradient was increased to a higher value, and the process repeated until final gradient. The required gradient was calculated using equation 30.

One Layer Sample

The prepared sample had a dry unit weight of 96 pcf and a porosity of 42%. Figure 38 shows the relationship between the conductivity and the applied hydraulic gradient. As the gradient increased from 70 to 117, the conductivity also increased by about 20%. This is because a gradient of 117 is probably not much greater than the maximum past preconsolidation pressure which allowed the sample to swell, thereby increasing the porosity and, hence, increasing the conductivity. As the gradient increased from 117 to 408, the conductivity decreased by about 44%. Beyond a gradient of

408 the conductivity stayed constant. The reduction in conductivity for higher gradient can be explained by the following discussion.

As the gradient increased above the past preconsolidation pressure, the soil sample started to consolidate (densification), which leads to lower porosity and, hence, lower conductivity.

As the gradient increased, the amount of dislodging and washing down, through the pore space, of fine particles will increase. The amount of dislodging and washing down will be more pronounced with low initial unit weight. This will decide how loose and the manner in which some fine particles are adhering to the internal surface of the pores. This process will result in the blocking of some pores, especially the ones located downstream.

As the gradient increases, water will flow to new locations within the sample which increases the involvement of more clay particles in the process of swelling. This will lead to the closing up of existing pore passages and opening up of new ones. These new ones may well be leading to dead-end pores, thereby reducing the effective porosity and, hence, lowering conductivity.

Three Layer Sample

The prepared sample had a dry unit weight of 105 pcf and a porosity of 34%. Figure 39 shows the relationship obtained

for this sample and is essentially the same as that shown in Fig. 38, except that the values have changed. As the gradient increased from 63 to 263, the conductivity increased from 2.45 to 6.57 * 10^{-8} cm/s. This is due the fact that this sample had a higher unit weight leading to a higher maximum past preconsolidation pressure (due to compaction), and, therefore, no reduction in pore volume has taken place but rather the soil is swelling. This swelling leads to higher porosity, resulting in higher conductivity. Above gradient 263 the samples start to consolidate, leading to a reduction in pore volume which will result in lower conductivity. All other discussions outlined for sample one applies to this sample.

However, the difference in the obtained relationship between the conductivity and the gradient for the two samples is entirely due to the fact that sample one had more uniform distribution of unit weight and porosity than those for sample two. This is due to the method of compaction as discussed before.

Conductivity vs. Unit Weight vs. Time

General

The effect of the different compaction methods and times on the hydraulic conductivity was studied using two samples that have the same thickness of 4.6 inches and were tested under the same hydraulic gradient of 70. These samples were

obtained using the same procedures used for sample preparation for suction tests outlined in Chapter 2. The first sample was compacted in accordance with ASTM D698A (standard light compaction), resulting in a dry unit weight of 99 pcf and a moisture content of 10.1%. The second sample was compacted in accordance with ASTM D1557A (standard heavy or modified compaction), resulting in a dry unit weight of 118 pcf and a moisture content of 11%.

First Sample

The relationship obtained between the conductivity and elapsed time is shown in Fig. 40. As can be seen, as the time from the start of the conductivity test increased from about 2 hours to 18 hours, the conductivity decreased by 40%. Generally, as the time increased, neglecting the slight irregularity in the measured conductivity between 18 and 26 hours, the conductivity increased to a maximum value of 29 * 10^{-7} cm/s at about 99 hours. Beyond this time the conductivity stayed constant. This general behavior is explained below.

The main reason is the dislodging and washing down of fine particle from the internal surface of the pore spaces and along the pore passages, respectively, during the early hours of the test. This process will be enhanced by the relatively low initial dry unit weight and moisture content of the sample. Low initial dry unit weight results in the

existence of loose fine particles on the internal surface of the pores, and higher porosity and larger pore spaces which facilitate the washing down of the loose fine particles. Low initial moisture content will reduce the attraction between fine particles that are covering the inner surface of the pores, resulting in a quick dislodging of those particles. These processes will lead to the blocking of pore spaces, thereby reducing conductivity.

Due to the relatively low initial moisture content, the clay particles upon the availability of more water will start adjusting their adsorbed water (double layer), reducing the size of the pore spaces, which results in low conductivity.

After the completion of the above processes after about 18 hours, then, the fine particles that were blocking the pore spaces will be washed out of the sample and or forced, by the high seepage forces due to the gradient, to adhere to the inner surface of the pore spaces. The flaky shaped clay particles tend to adhere to the inner surface of the pores, with the long axis being perpendicular to the seepage forces. These processes will lead to higher conductivity.

Second Sample

The obtained relationship between conductivity and elapsed time for this sample is shown in Fig. 40. Due to the relatively higher initial dry unit weight and moisture

content, the conductivity of the sample did not vary significantly with time.

Moisture Content Distribution After Conductivity Tests

The distribution of moisture content with depth along the sample after the completion of conductivity tests was studied for a number of samples. This is performed by dividing the samples, after the completion of the conductivity tests, into four equal parts. Then, one part is selected and divided into equally thick pieces. The thickness of these pieces varied with the total thickness of the tested samples and generally ranged from .5 to 1 cm thick. The moisture content of these pieces, then, is determined following the ASTM D2216 method of testing (ASTM 1989). The results are plotted such as those shown in Figs. 41 and 42. From close observation of these two figures, the following can be stated and deduced.

1. After and during the conductivity tests the distributions of moisture content with depth along the sample are not uniform as currently assumed by various researchers. This is due to the nonuniformity of the unit weight and en-homogeneity of the soil within the tested sample. This will result in nonuniform pore sizes and distribution which will lead to variable moisture distribution.

2. The variations in the moisture content are highest at the upper part of the soil sample. This is due to the

fact that the upper parts are closer to the applied hydraulic gradient. This will lead to the upper part being subjected to the highest seepage forces, and, hence, the soil in these parts will be more saturated than those in the lower parts. Figure 41 shows some deviation from this condition. This is due to the possibility that the bottom surface of the soil samples were not dried appropriately in order to remove surface water.

Laboratory Desiccation Tests

Two 18-inch-thick soil samples were prepared in the laboratory to study the effect of soil desiccation on the hydraulic conductivity, temperature distribution, and the moisture content profiles before and after conductivity tests. The two samples were prepared in accordance with the procedures discussed in Chapter 2.

Temperature Distribution

The thermocouple temperature readings were recorded every day for 16 days. Then, an average temperature per each thermocouple was calculated. These average temperatures together with the depth of the respective thermocouples are shown in Table 5. As can be seen, the temperature along the sample varies by a maximum of 3.5°F and the temperature at the top 2 inches of the sample is the lowest. This is due to the close proximity of the upper part to the atmosphere.

Hydraulic Conductivity Test

After 16 days in 100°F temperature, hydraulic conductivity tests under a hydraulic gradient of 70 were performed on one of the samples in order to establish its hydraulic conductivity variation with time. The results are shown in Fig. 43. For the first 20 hours the water flow was very high due to high suction which will result in high infiltration. After 21 hours the water started flowing close to saturated flow conditioned, and, after 45 hours, the conductivity dropped by 74%. After this time the conductivity value did not fluctuate significantly and remained at about $2 * 10^{-7}$ cm/s. This behavior is mainly due to the reduction in pore spaces as the clay particles start taking up the inflowing water and adjusting (expanding) its double layer boundaries.

Moisture Content Distributions Before and After Conductivity Tests

The moisture content distribution of the two samples after 16 days of desiccation was determined following the same procedures discussed above. The sample which contained the thermocouples was used to determine the moisture content distribution before the conductivity test, while the other sample was used to obtain the moisture distribution after the conductivity test. The results are plotted and shown in Fig. 44. The figure shows the following:

1. In the upper 8 cm both samples displayed the highest variations in moisture content. This is due to the fact that this part of the sample, before the conductivity test, is subjected to higher evaporation than any other part. For the sample used for the conductivity tests, this part was closer to the applied hydraulic gradient and, hence, has a higher moisture content.

2. Below 8 cm the moisture content distribution of the sample tested for conductivity was uniform. The other sample displayed uniformity below 24 cm.

3. The average moisture content before and after the conductivity tests was 21 and 27%, respectively. This means that after a moisture content increase of only 6% the soil possessed a stable and usual value of conductivity.

TABLE 5. Average Temperature vs. Depth Along Soil Sample

Depth of Thermocouple (inches)**	Average Temperature (°F)*
0.5	121.23
1.5	122.98
3	124.43
6	124.75
10.5	124.77
16.5	124.73

*Average readings were taken over 16 days at 100°F.

**Total soil thickness is 18 inches.

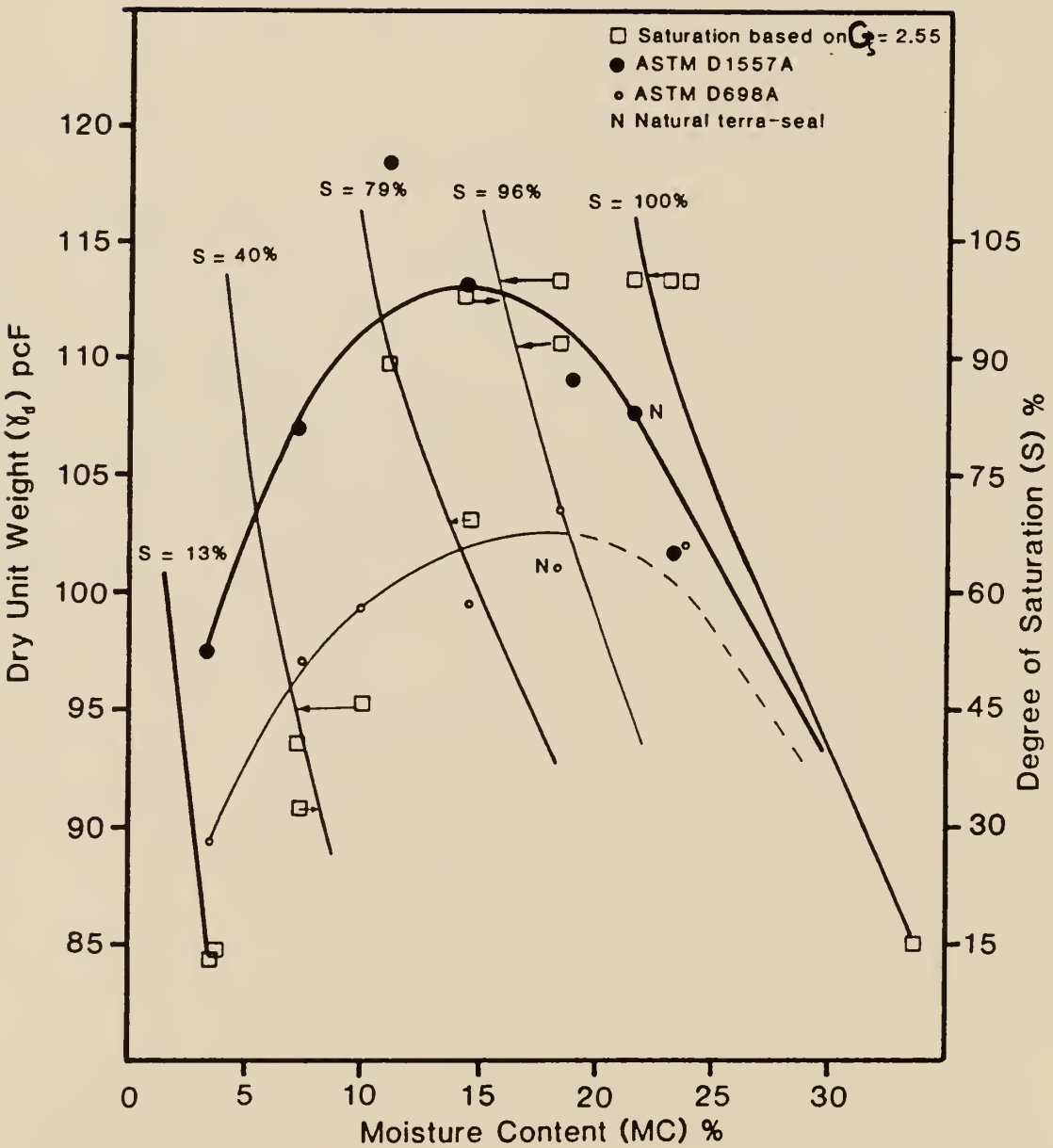


Fig. 31. Degree of Saturation vs. Dry Unit Weight vs. Moisture Content.

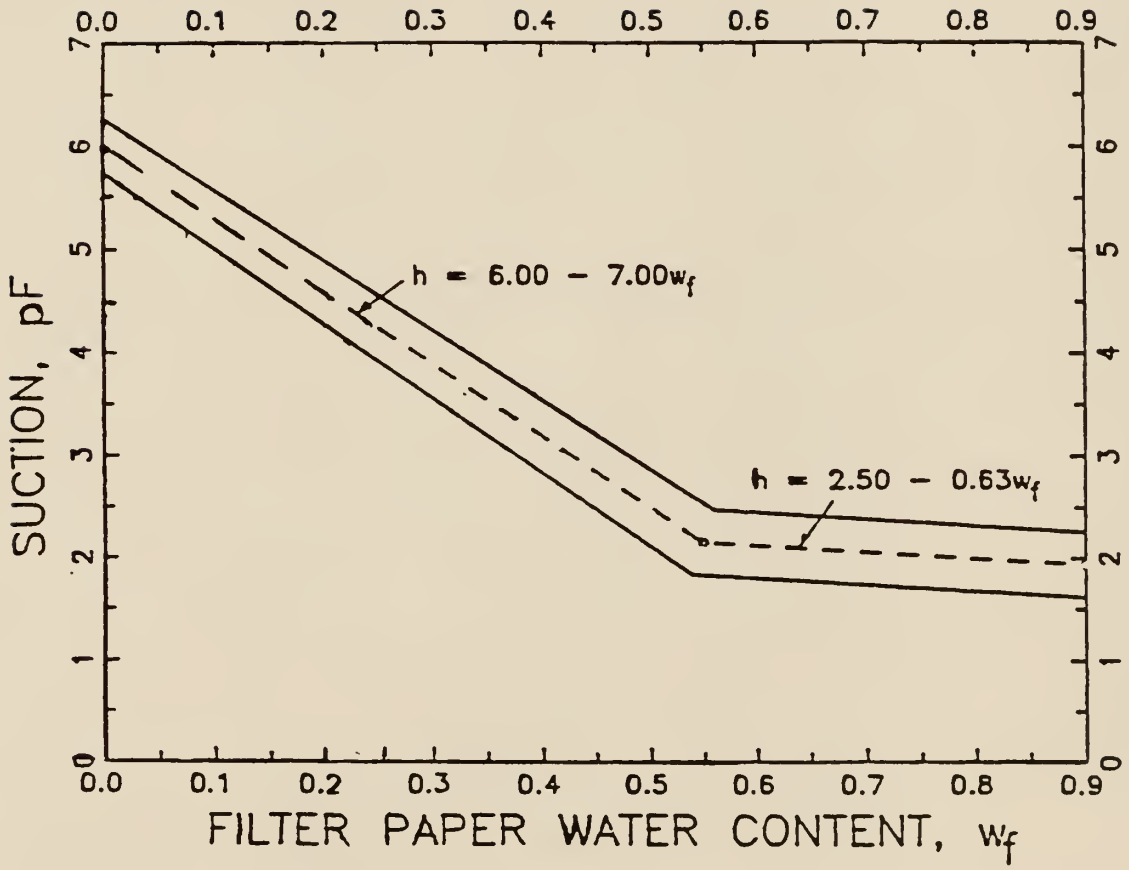


Fig. 32. Suction vs. Filter Paper Water Content (McKeen 1988).

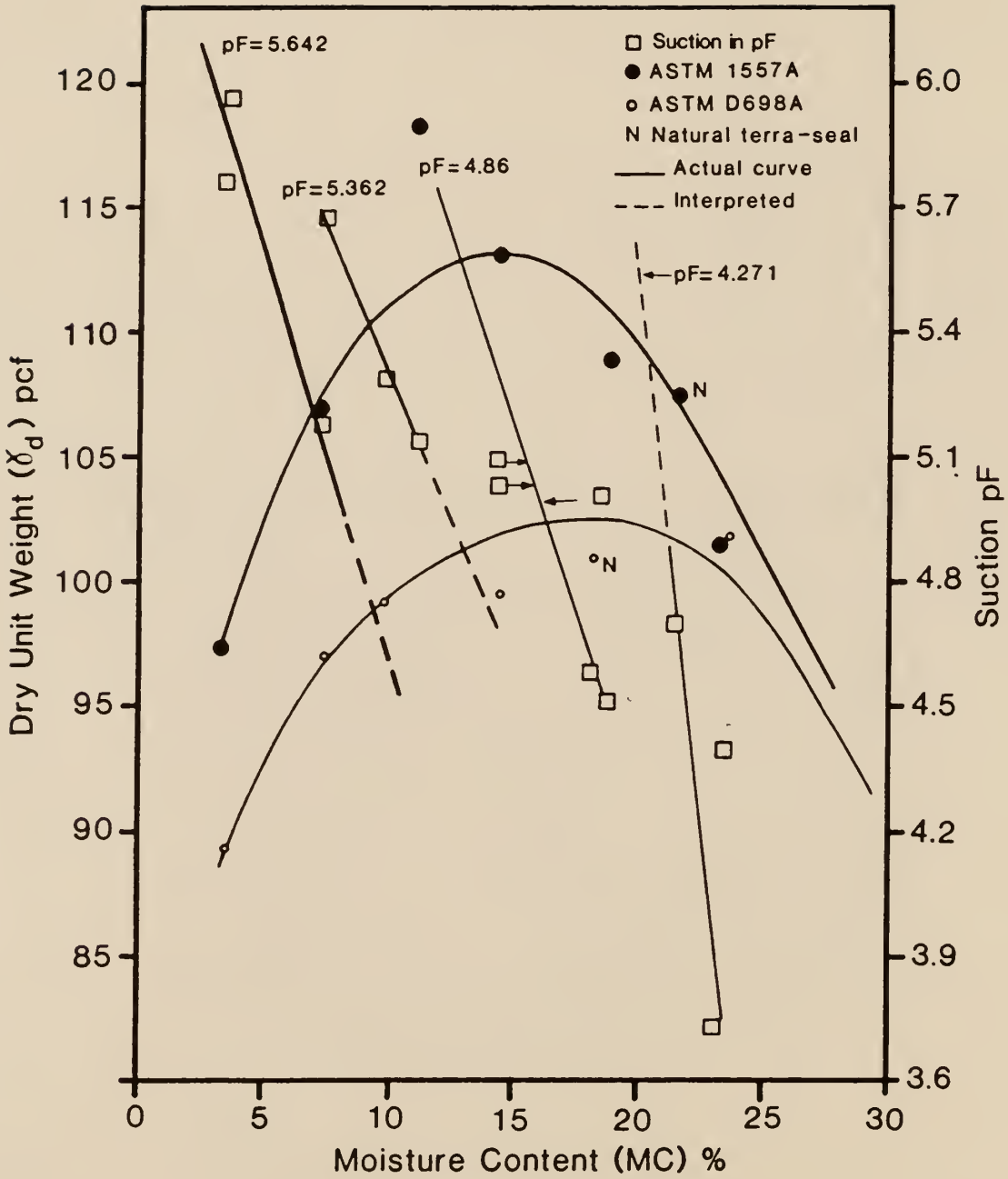


Fig. 33. Soil Suction vs. Dry Unit Weight vs. Moisture Content.

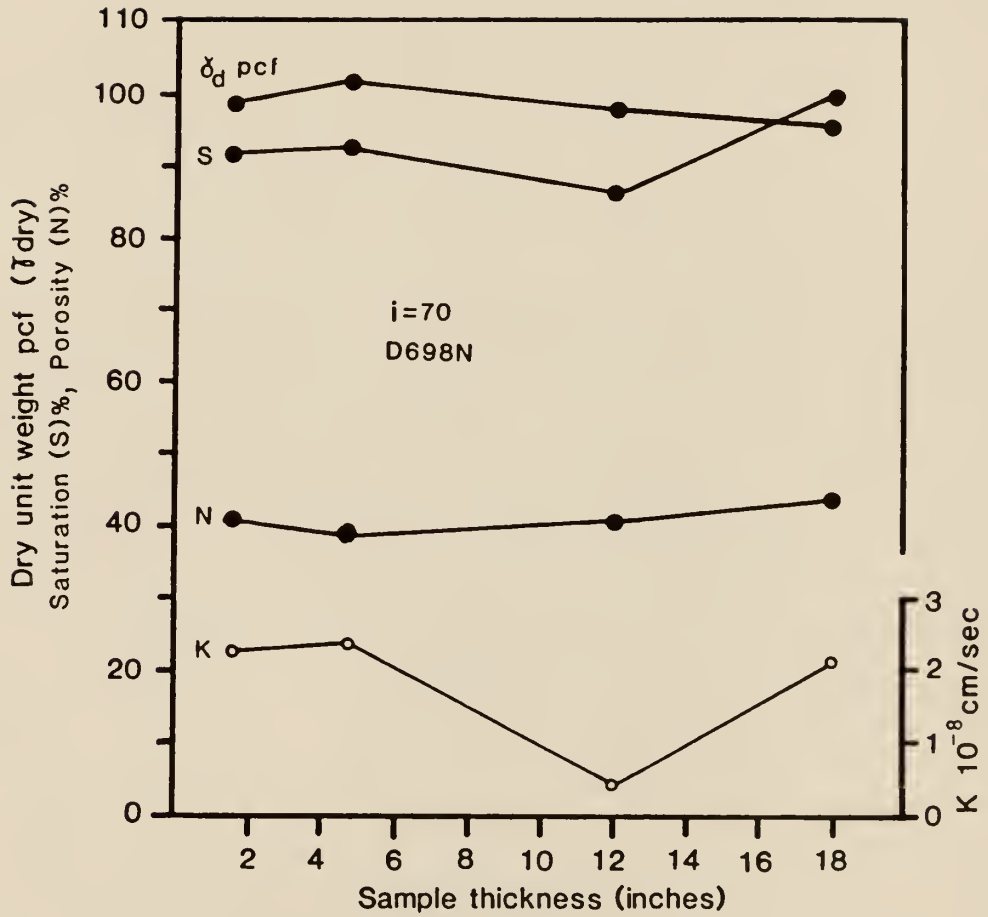


Fig. 34. Hydraulic Conductivity, Dry Unit Weight, Saturation, and Porosity vs. Sample Thickness.

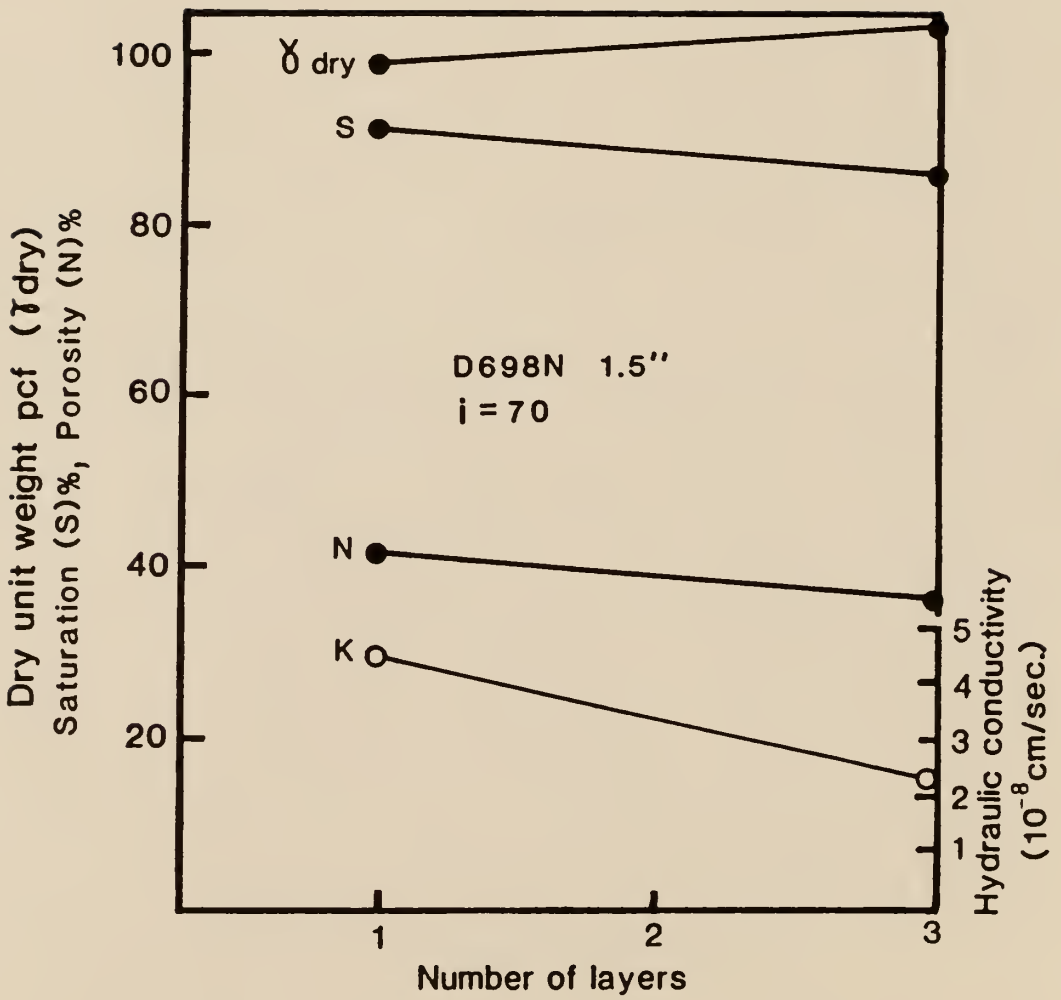


Fig. 35. Hydraulic Conductivity, Dry Unit Weight, Saturation, and Porosity vs. Number of Layers for 1.5" Sample.

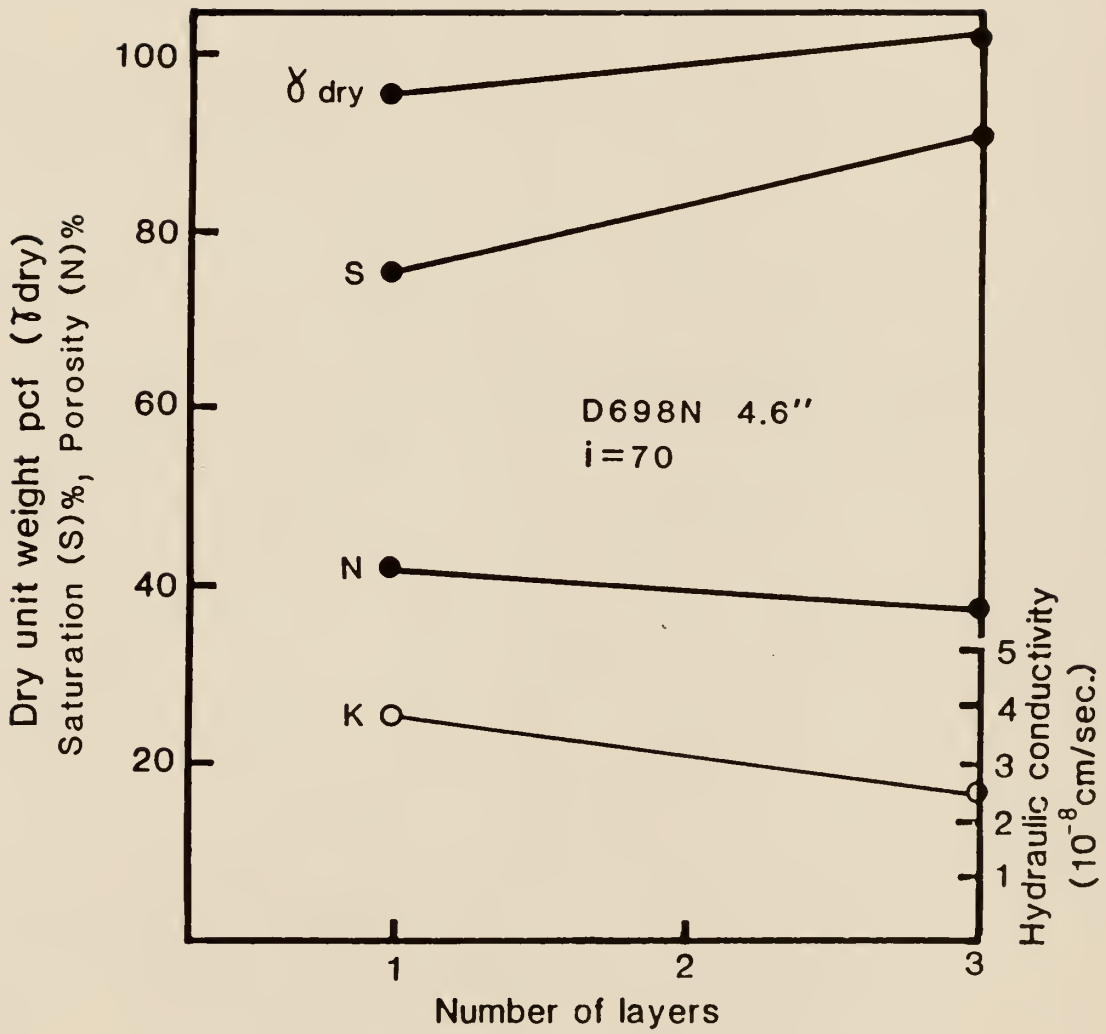


Fig. 36. Hydraulic Conductivity, Dry Unit Weight, Saturation, and Porosity vs. Number of Layers for 4.6" Sample.

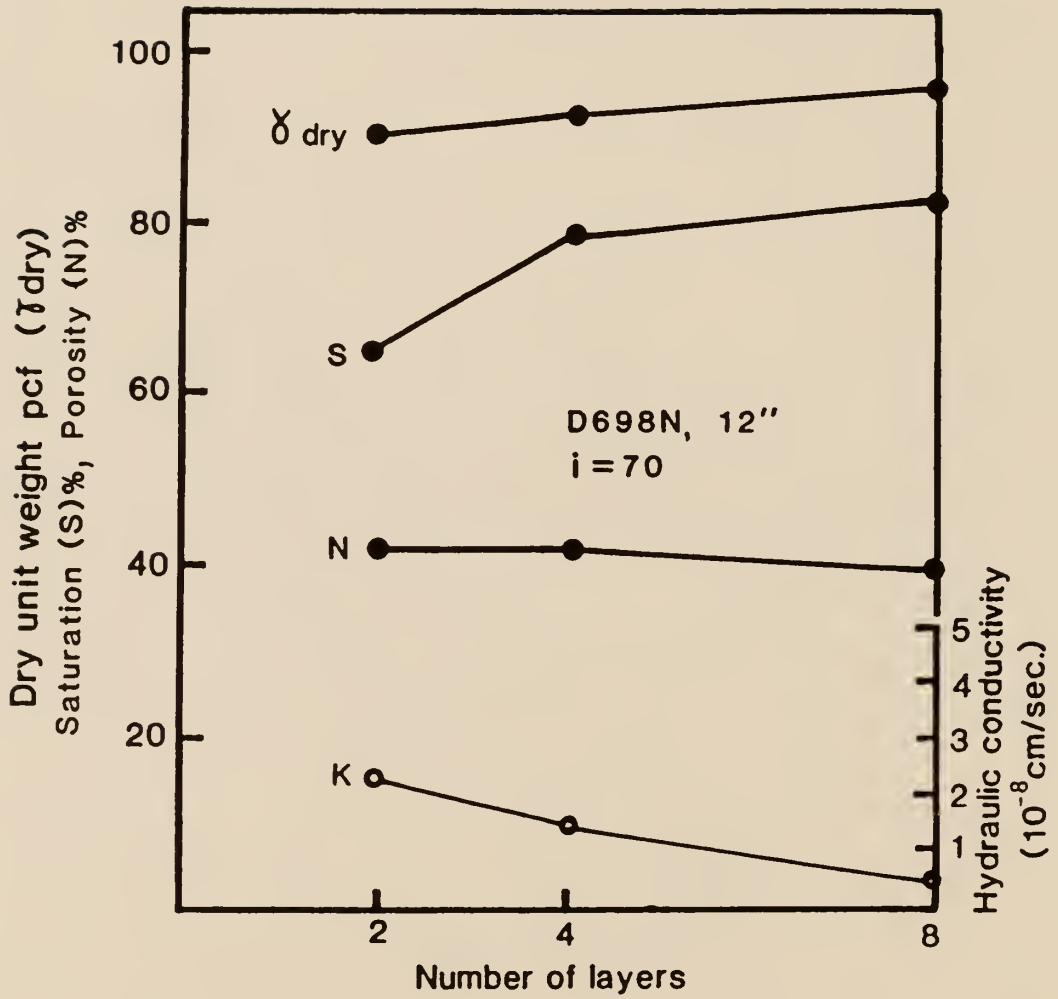


Fig. 37. Hydraulic Conductivity, Dry Unit Weight, Saturation, and Porosity vs. Number of Layers for 12" Sample.

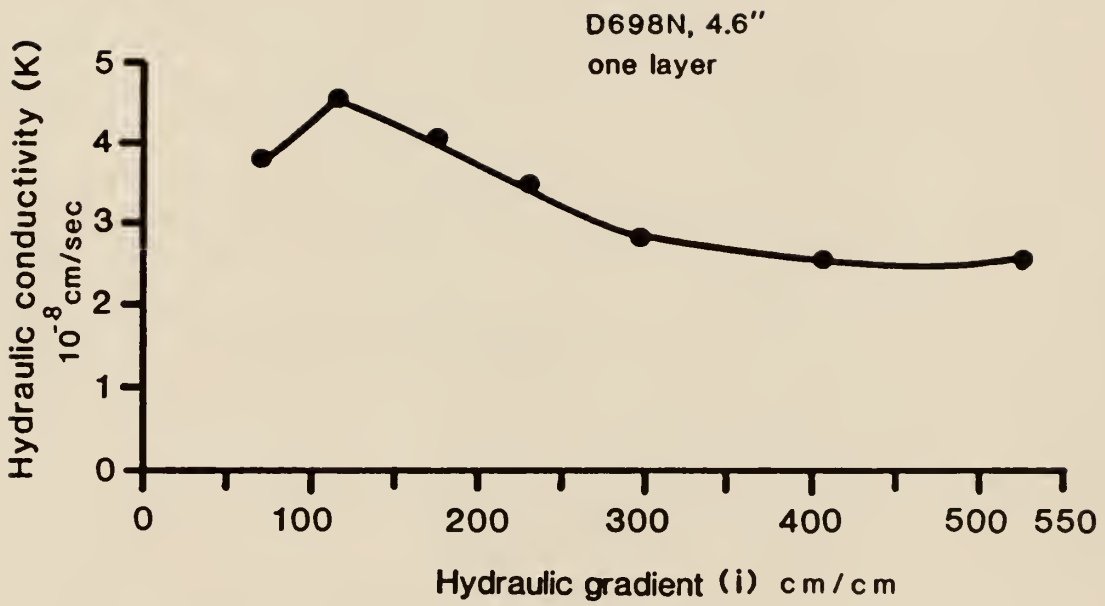


Fig. 38. Hydraulic Conductivity vs. Hydraulic Gradient for 4.6" One Layer Sample.

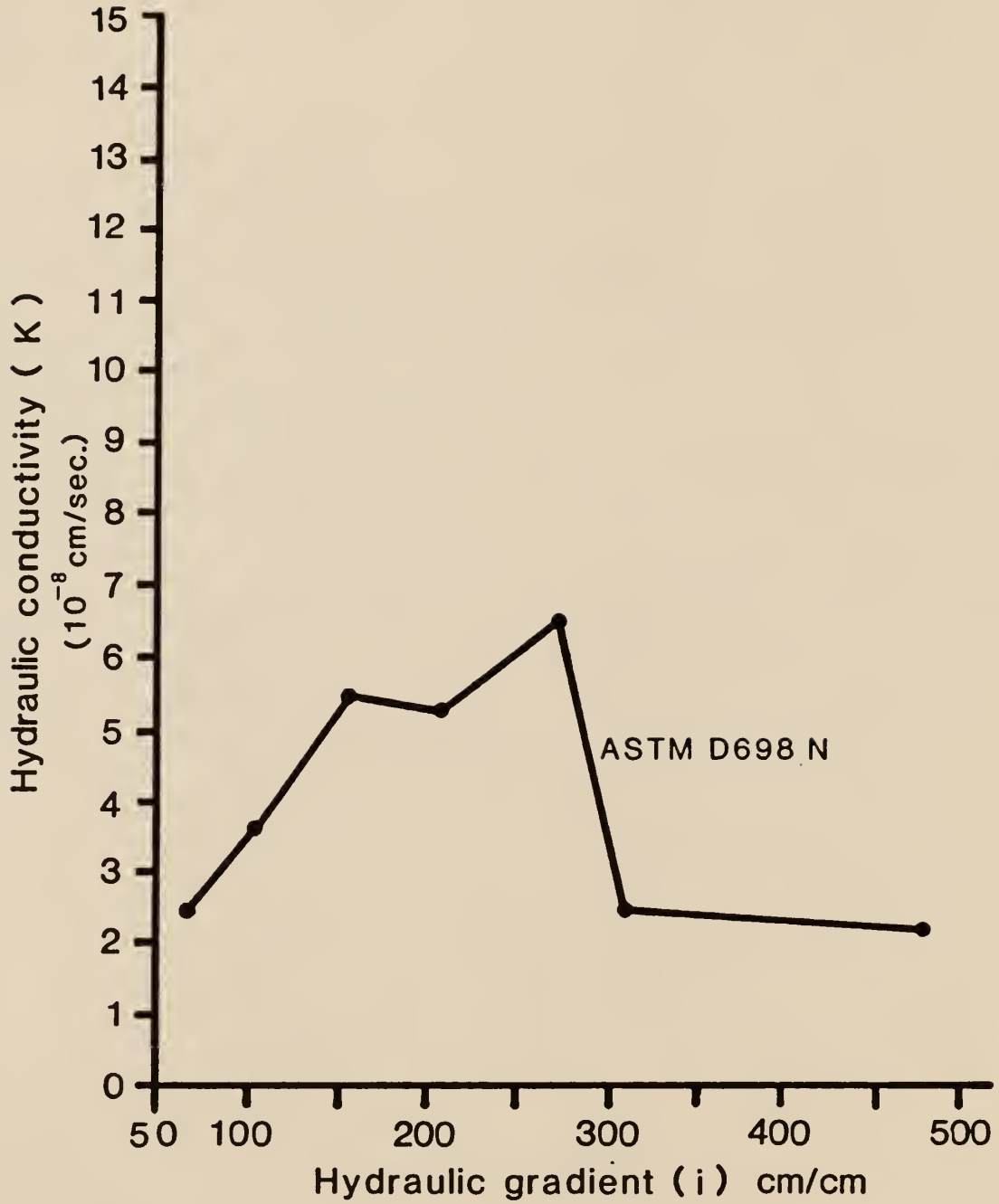


Fig. 39. Hydraulic Conductivity vs. Hydraulic Gradient for 4.6" Three Layer Sample.

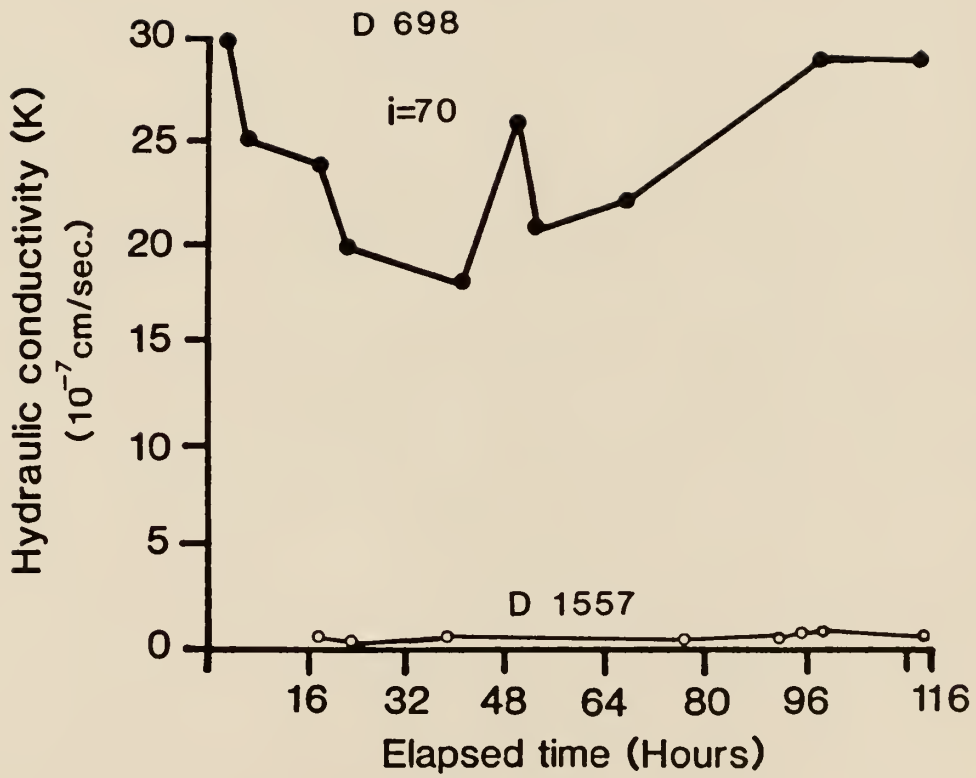


Fig. 40. Hydraulic Conductivity vs. Elapsed Time.

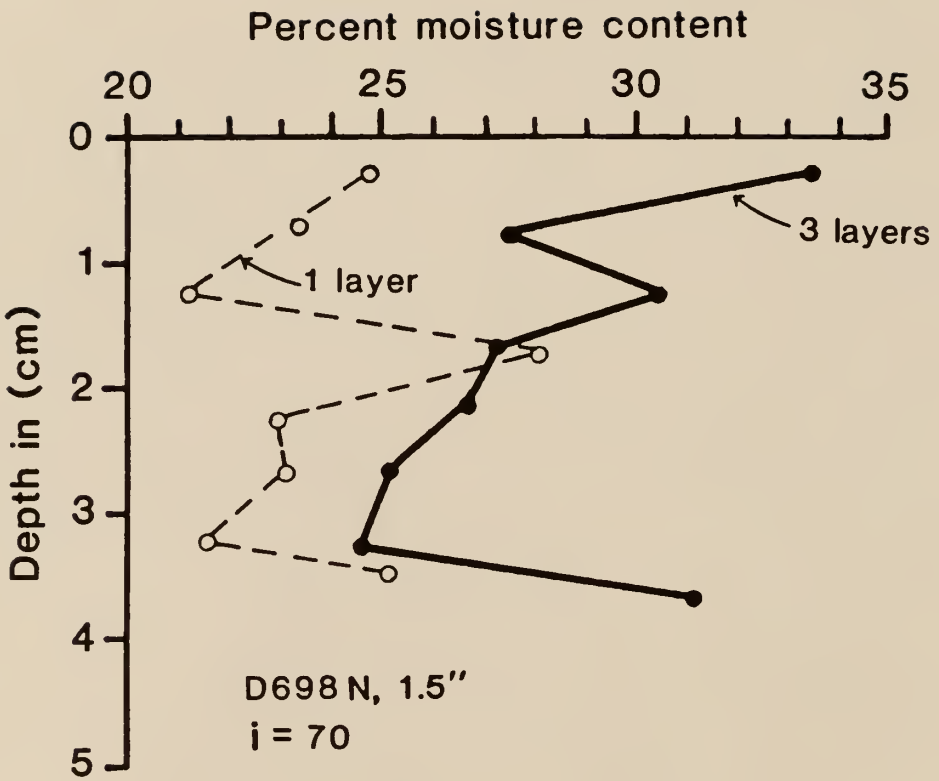


Fig. 41. Moisture Content vs. Depth of 1.5" Sample.

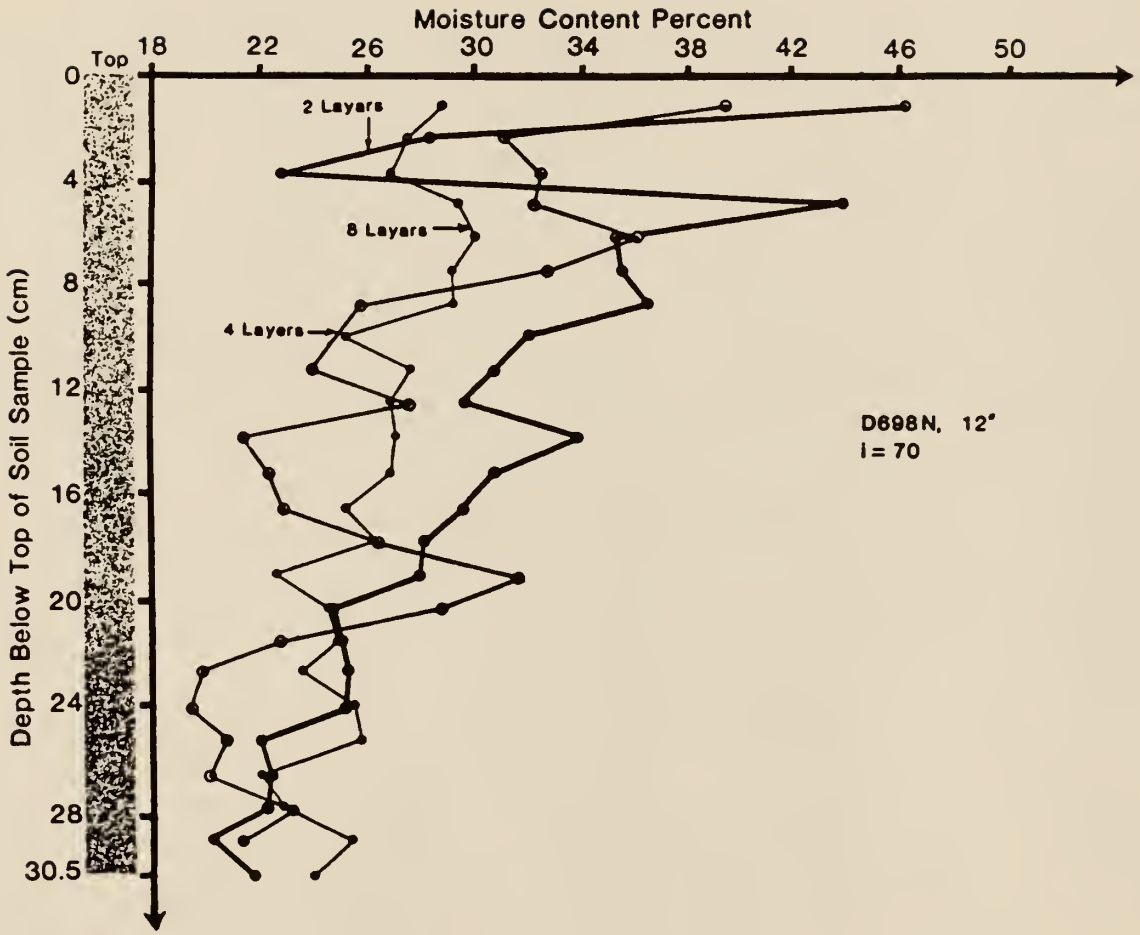


Fig. 42. Moisture Content vs. Depth for 12" Sample.

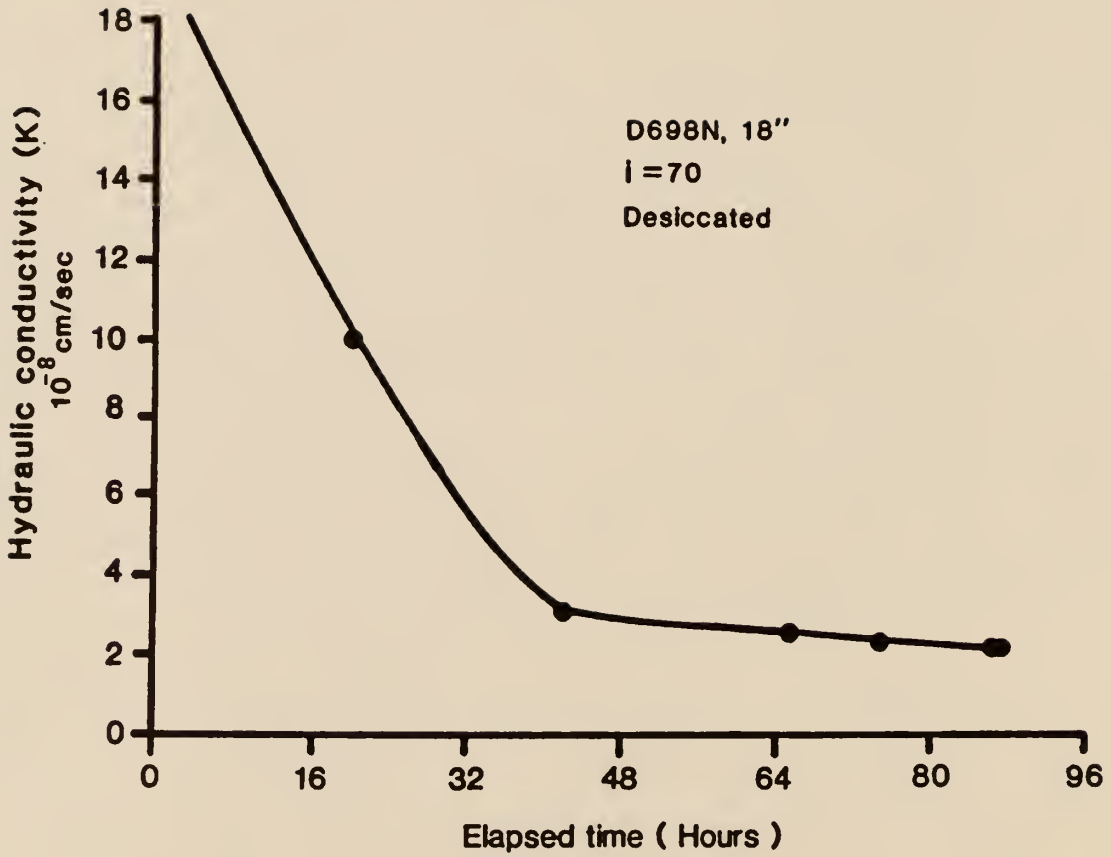


Fig. 43. Hydraulic Conductivity vs. Elapsed Time for Desiccated Sample.

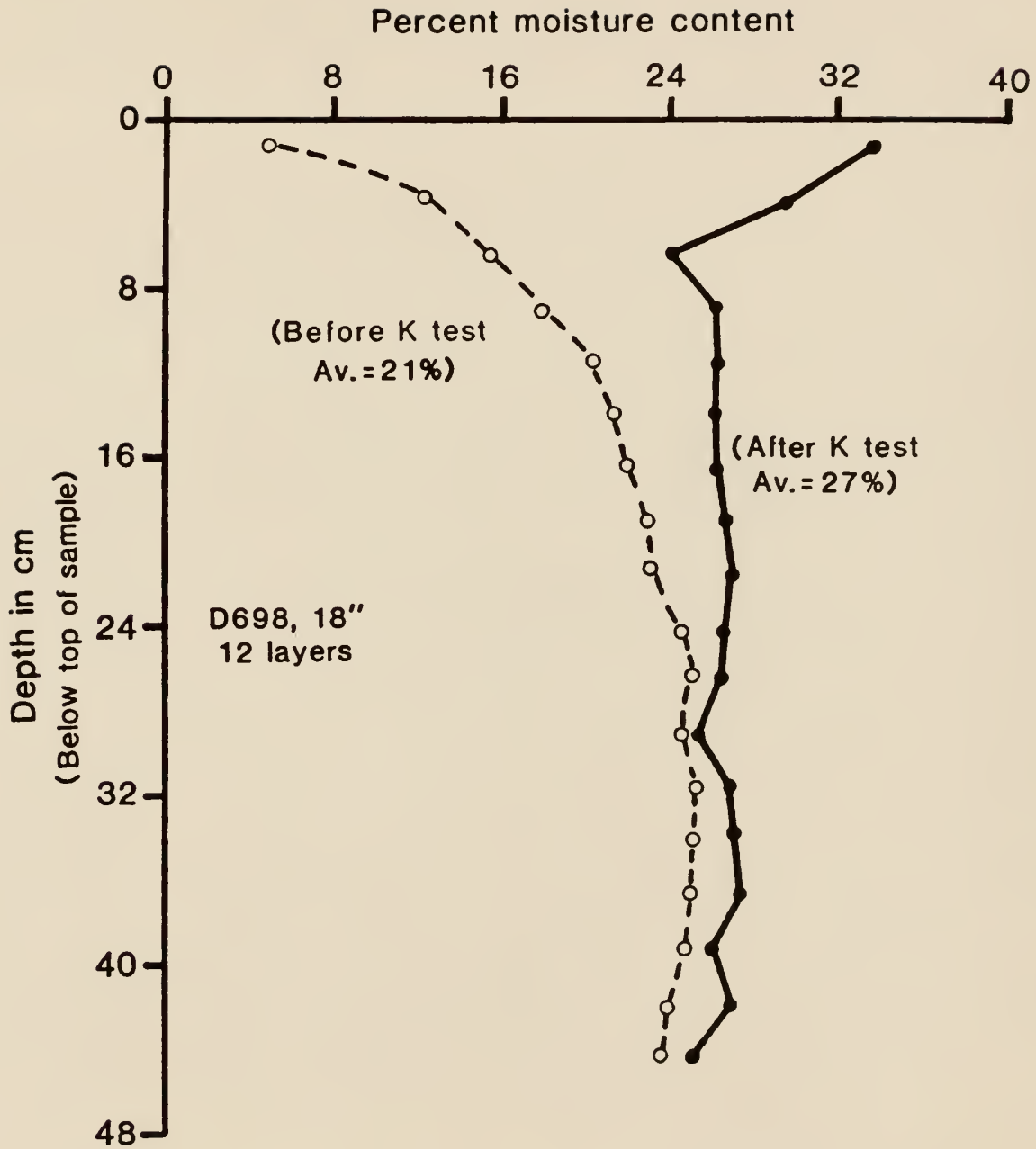


Fig. 44. Moisture Content vs. Depth for Desiccated Sample Before and After Hydraulic Conductivity Test.

CHAPTER 4
FIELD WORK, RESULTS, AND DISCUSSION

Field Infiltration Tests

General

A total of 10 field infiltration tests were performed on two clay landfill liner projects. These projects are the South West Alachua and Astatula located in Archer and Astatula, respectively, in Florida. All equipment utilized in the field infiltration testing was newly designed by the author, was not copied from nor did it resemble any existing field testing methods, and was build on the campus of the University of Florida. Furthermore, the methodology and the concept of the performed field infiltration tests also were new.

Methodology and Concept of the Field Infiltration Tests

The first important concept on which the performed field infiltration tests were based was that the field tests must be run in a similar manner to those performed in the laboratory. This is very important since the field conductivity is compared to that of the laboratory conductivity in order to determine the scale effect and the

effect of sample disturbance on the laboratory conductivity value. In the laboratory conductivity tests (rigid and flexible wall type), the water flow is allowed to travel through the soil sample in the vertical direction only (no lateral flow is allowed). The length of flow traveled is equal to the length of the soil sample (L), and the sample is near full saturation when conductivity is measured. In the existing field hydraulic conductivity and infiltration testing water is allowed to travel laterally, as can be seen in Figs. 10 and 27, since the water is allowed to travel laterally, the exact length of the soil sample is not known but is usually taken as equal to the depth of embedment of the central ring (D), as is shown in Fig. 25. The tested soil (soil located within the distance D) is assumed to be fully saturated which is a very inaccurate assumption, as is shown by Fig. 27.

On the other hand, in the performed field infiltration tests a full section of the clay liner is isolated by pushing the sampler (Fig. 30) through the full thickness of the liner. This allows the water to flow in the vertical direction only, i.e., no lateral flow is allowed, and the tested length of the soil is well defined, i.e. equal to the length of the sampler (Fig. 30). Since the performed field tests were infiltration tests and not conductivity tests, then, the tested soil, within the sampler, was treated as a

partially saturated soil (which is the case always), and hence, there was no need to assume it to be fully saturated.

The second important concept on which the performed field infiltration tests were based was that it must be field rugged, rapid, and simple to operate. The field conductivity tests are done as part of a quality control and a quality assurance program designed to insure that the compacted clay meets the designed hydraulic conductivity value set in the project specifications and similar to that measured in the laboratory. Therefore, field testing must be rapid (so a number of them can be performed on each compacted clay layer), must not delay or hold up field construction, must be able to be performed on the actual liner with a minimum amount of disturbance to the compacted liner, must be simple enough to be conducted by an engineering technician as any normal field control tests, must be field rugged and insensitive to errors due to field handling, and must not involve tedious calculations.

However, all existing and suggested field hydraulic conductivity and infiltration test methods are messy (require a large area and mixing of bentonite); require long time periods to set up; require highly technical personnel to perform, as is evident by Fig. 11; are highly sensitive to errors due to field handling, as can be seen by Figs. 10 and 11, require a long time to saturate the soils located within the distance D (Fig. 25) so that the saturated hydraulic

conductivity is measured; and involve lengthy calculations. The aforementioned items are exactly the reasons why no field hydraulic conductivity or hydraulic infiltration tests are performed or included in the project specifications of any currently constructed clay liners.

On the other hand, the field infiltration tests performed in this study required half an hour to set up and 1 to 4 hours to complete (since only the hydraulic infiltration values were measured and not the saturated conductivity which will take a long time to measure), were not messy at all since each test resulted in a hole 4.5-inches in diameter in the clay liner which was filled up with clay and compacted with a minimum amount of difficulty, did not involve any sensitive equipment so they were rugged and relatively insensitive to errors due to field handling, did not involve lengthy formulas or calculations, and did not interfere with any field operations. Figure 45 shows a typical field hydraulic infiltration set up with all the parts and dimensions clearly marked.

The third important concept that the performed field infiltration tests were based on was that there must be a check system by which the predicted saturated hydraulic conductivities, obtained based on the field infiltration values, are verified by laboratory tests performed on the same clay samples that were tested in the field. This is very important because once a field conductivity test is

performed (usually just one test is performed due to the factors discussed earlier), there will be considerable attention given to the results. Also, conclusions will be drawn regarding the integrity of the compacted liner, type of clay used, and method of construction once the predicted conductivity value meets the specifications or once the field tests favorably compare to laboratory tests. If this happens, many accusations could develop about everybody concerned with the project in hand. This was demonstrated by the results of the field conductivity tests obtained by Daniel (1984). Therefore, the clay supplier, the construction company, and the landfill industry in general need a check system of the field predicted conductivity values. All existing suggested methods of field infiltration and conductivity testings do not incorporate such a check system of the predicted field values.

On the other hand, the performed field infiltration tests were designed such that once the field infiltration test was terminated, the sampler together with the plastic tube that contained the tested clay sample was retrieved. This can be seen in Figs. 30 and 45. Then, the plastic tube containing the clay sample was removed from the sampler, brought back to the laboratory with minimum amount of disturbance, and a laboratory hydraulic conductivity test was performed in a similar manner to any laboratory test. This test was considered as a laboratory test that was performed

on a field-obtained undisturbed sample, and, therefore, the predicted conductivity value was treated as a very credible check on the predicted conductivity value that was obtained based on field infiltration test.

Information Obtained Prior to the Start of Field Testing

Prior to the start of field testing, the thickness of the clay liner to be tested was obtained. This was necessary for the selection of the appropriate length of sampler so the sampler could be driven into the sandy subgrade below, as can be seen in Fig. 45. This insured that there was no lateral flow of water to the surrounding clays, as discussed above. Other information that was gathered, whenever available, was field dry unit weight and moisture content in close proximity to the performed field infiltration tests. This was necessary for later comparison between the various values.

Field Infiltration Setup

The steel sleeves containing the plastic tubes,4 inches in diameter, were pushed into the compacted clay liner by an on-site backhoe. The first field infiltration test at each project was pushed in by hand jacking. This was done to obtain the effect of the driving method on the measured unit weight and the hydraulic conductivity of the clay sample. Due to the adhesion and friction that developed during the driving of the sampler between the clays and the inner

surface of the plastic tube in the sampler, the clay inside the plastic tube heaved up, as it indicated in Fig. 45. Then, a layer of coarse sand was placed on top of the clay inside the sampler. This was done to prevent the floating of clay particles that were located close to the surface of the clay in the sampler after wetting. An acrylic plastic tube 2.1 inches diameter was tied down to the sampler by four bolts. Water was then poured inside the acrylic plastic tube to within 0.5 inch of the top. The acrylic plastic tube was extended through a plastic cap cover to a plastic tube which was 0.25 inch in diameter. This plastic tube was mounted on a yard stick in order to monitor the level of water inside the plastic tube. This small plastic tube was then filled with water to within 1 to 2 inches from the top. All interfaces were covered by a generous amount of silicon sealant prior to the addition of water in order to prevent any leakage or evaporation from the system. A cross section of the this setup is shown in Fig. 45. There was no leakage noticed in any of the tests performed. All visible entrapped air bubbles in the system were removed by gentle sliding of a thin wire up and down the small plastic tube. This process represented the longest part in the field test setups. Then, the levels of water inside the small plastic tube at an arbitrary length of time were recorded. The water levels and the time were recorded for several days for each test.

South West Alachua Landfill-Top Cover

General

The first two field infiltration tests were performed on the top cover (liner cap) of the existing south west Alachua landfill located in Archer, Florida. The project location and vicinity map is shown in Fig. 46. This landfill was constructed during 1986 as a nonhazardous class I and III solid waste landfill. A class I and III landfill contains solid wastes that are generated by domestic and commercial sectors. The project is approximately 32 acres in area and was constructed as an 8-inch-thick single layer cap or cover for the compacted solid wastes. The project specifications required a minimum saturated hydraulic conductivity of $5 * 10^{-7}$ cm/s to be measured in the laboratory using the flexible wall permeameter. Full details of the project specifications are included in Appendix C. There was no field hydraulic infiltration or conductivity performed on the cap liner. All laboratory (conductivities and index properties) and field (unit weights and moisture contents) testings were performed by a professional testing laboratory, Ardaman and Associates, located in Orlando, Florida. Full details of most of the field and laboratory test results are included in Appendix C.

Field Infiltration Tests

A 4- by 10-foot arbitrary location on the top of an existing capped section of the landfill was chosen for the field infiltration tests. The clay cap was exposed by removing about 3 to 3.5 feet of top soil and a sandy layer. Two field infiltration tests were performed on the exposed clay soil at the locations shown in Fig. 47.

Readings of the water levels (H_{o1} and H_{o2}) in the small plastic tubes at the start of the tests and after 4.5 hours and 5 days were recorded for both infiltration tests. Then, by using equation 11 the coefficient of hydraulic infiltration at both locations was calculated. These values are shown in Table 6. After the termination of the tests, the soil samples from both locations were brought back to the laboratory.

Laboratory Tests

In the laboratory, saturated laboratory hydraulic conductivity tests were performed on the sample obtained from field test location number 2. The other sample was disturbed during extrusion and, therefore, had to be discarded. The sample was tested under a hydraulic gradient equal to 70, and the stabilized saturated conductivity was calculated using equation 2. The predicted conductivity value was $7.7 * 10^{-8}$ cm/s, as indicated in Table 6.

In the laboratory and prior to any other tests, the unit weight and moisture content of the two soil samples were measured and values are shown in Table 6. The unit weight and moisture content for test number 1SH (short sleeve) and 2LA (long sleeve with angled cutting shoe) were 105 and 106.8 pcf and 24.6% and 20.6%, respectively.

Conductivity Using Suction and Infiltration Values

The above unit weights and moisture contents were used in Fig. 33, and suction values (H_s) for the two soil samples were obtained. These value of H_s were used in equation 34, which is a modified version of equation 2 (Taylor 1948), together with the infiltration reading after 4.5 hours, and the saturated hydraulic conductivity of both samples was predicted:

$$K = (Q * L)/ (A * T * [H_o+H_s]) \qquad (34)$$

where all symbols are the same as those for equation 2.

The predicted values of the conductivity for sample 1SH and 2LA were 4.7 and 3.3 * 10^{-9} cm/s, respectively. These values are indicated in Table 6.

Comparison Between the Predicted Conductivity Values

All laboratory and field test results obtained for both field tests are included in Table 6 together with all the formulas used in the calculations. Laboratory conductivity test results that were measured by Ardaman and Associates together with the corresponding unit weights and moisture contents are also included in the table. These conductivity values were selected, among many test results, by selecting the most similar unit weights and moisture contents to that measured by the author. Details of the Ardaman test results are included in Appendix C. However, Ardaman and Associates test results were a hydraulic conductivity of $7.4 * 10_{-8}$ to $2.1 * 10_{-9}$ cm/s, dry unit weight of 89.4 to 111.9 pcf, and a moisture content of 17.7% to 29.8%. As can be seen from the table, the Ardaman values compare very well with those predicted by the combination of the suction and the infiltration values. Also, the conductivity value predicted in the laboratory by the author using the rigid wall permeameter compared well with the high end of the Ardaman range. Furthermore, the unit weights and moisture contents measured by the field sampler also compares very well with those measured by Ardaman and Associates.

Astatula Ash Residue Monofill-Bottom Liner

General

The project is located on Highway No. 561 in Astatula, Lake County, Florida. The project location and vicinity map are shown in Fig. 49. The project is approximately 6 acres in area and was constructed between May and July of 1990. This project was classified and constructed as a hazardous solid waste landfill to house hazardous bottom ash residue that resulted from the burning of some solid wastes. A total of eight field infiltration tests were performed at this project. The first five of these tests were performed on three test strips that were constructed at the project site and before the construction of the actual landfill project. The last three field infiltration tests were performed on the western evaporation basin which is part of the actual landfill. The location of the test strips, landfill, and evaporation basins as well as all field infiltration tests are indicated on Fig. 50.

The clay liner at this project was a minimum of 12 inches thick constructed in three 4-inch-thick lifts. The project specifications required a maximum saturated hydraulic conductivity of $1 * 10^{-8}$ cm/s to be measured in the laboratory, using flexible wall permeameter, on field-obtained samples. Full details of the project specifications are included in Appendix D. There were no requirements for the field hydraulic infiltration or conductivity tests to be

performed on the compacted liner, and there were none performed.

All laboratory (conductivities and index properties) and field (sampling, unit weights, and moisture contents) testings were performed by two professional testing laboratories, Ardaman and Associates and Jammal and Associates, located in Orlando, Florida. Full details of most of the field and laboratory test results are included in Appendix D.

Field Test Strips

Construction

A total of three 8- by 10-foot field test strips were constructed at the start of the project construction. The test strips were constructed in order to perform various field studies prior to the construction of the clay liner, the main study being the effect of number of layers placed on the unit weight and moisture of the resulting clay liner. These test strips were constructed using the same "Terra-Seal Natural Premix®" as that used in the construction of the actual landfill and this study. The test strips were constructed such that the total thickness and the amount of applied compaction energy were the same for all three test strips. The only difference between them was that each test strip was placed in a different number of equal layers. Test strip number 1 was constructed in three equal layers, which

was similar to the specified construction method of the clay liner. Test strip number 2 was constructed in one layer with a thickness equal to the total thickness. Test strip number 3 was constructed in two equal layers. The total compacted thickness of each test strip was about 8.5 to 9 inches, and the total compaction energy applied to each test strip was 16 passes of a 12.4-ton sheepfoot vibratory roller known locally as Dynapac. The general location of the these test strips are shown in Fig. 50. Figure 51 shows details with approximate dimensions of the test strips and all other field tests performed on them.

Unit weight and moisture content distributions for the three test strips

The dry unit weight and moisture content distributions for the three test strips were studied by obtaining an undisturbed soil block sample from each test strip. The approximate locations and dimensions of all such blocks, for each test strip, are indicated on Fig . 51. These blocks were obtained by hand excavating a 3-inch-wide trench that extended to the subgrade sandy soils (through the full depth of the test strips). The trenches were excavated around three sides of each block with the fourth side being broken easily upon lifting of the blocks, and then the blocks were transported to the on-site laboratory.

In the laboratory all three blocks were tested by following the same procedures in which the outer 6 inches of

soil were discarded first. This was done in order to remove the highly disturbed soils due to excavation, stress relaxation, and handling. Then, two arbitrary columns (A and B) were selected within the remaining section of each of the soil blocks. The soils along each column were then divided into approximately 1-inch-thick small blocks. These small blocks were weighed in air, covered with a known specific gravity wax, weighed again in air, and then dropped in a graduated cylinder that was filled with water, and the submersed volumes were obtained. From this information the wet unit weight of those small soil blocks were determined. Prior to waxing, a small part was taken from each small soil block and used for the determination of the moisture content. Using the moisture content values together with the wet unit weight, the dry unit weight of the small soil blocks were calculated. Then, by combining the measured values for the small soil blocks in column A and B, an average dry unit weight and moisture content were obtained for each depth along the soil block. Figure 52 shows plan views and cross sections of typical soil blocks with some of the above methodology and the associated dimensions.

The average values of dry unit weight and moisture content were plotted against the depth along the soil block of each test strip. These plots are shown in Figs. 53 and 54. Figure 53 shows the variations of the average dry unit weight of the compacted clay with depth for the three test

strips. It can be seen from Fig. 53 that the largest and lowest values of unit weight occur at the top 1 inch and lowest 3 inches, respectively. This is expected since soil located within the top 1 inch was subjected to relatively higher amounts of evaporation and surface compaction, while the soil located near the bottom was not compacted properly due to the loose nature of the subgrade sandy soils. Between the aforementioned depths the variations in the values of the unit weight were within about 5 pcf or less for all three test strips. However, test strip number 2 (constructed using one lift) showed the least amount of variation. This is because the clays in test strip number 2 were subjected to the same and uniform compaction energy, while the clays in the other two test strips were subjected to different amounts of compaction energy because they were placed in layers. Consequently, these test strips have nonuniform unit weight distribution. The overall average of the dry unit weight for test strip 1 (3 lifts), 2 (one lift), and 3 (2 lifts) was 77.9, 78.7, and 75.8 pcf, respectively.

The above analogy can be applied to the distributions of the average values of the moisture content with depths as can be seen in Fig. 54. This is because at moisture contents below the optimum values, the unit weight and moisture content are directly related to each other. The overall average moisture content for test strips 1, 2, and 3 was 34.8%, 33.4%, and 34.6%, respectively.

The above results demonstrate that placing the clay liner in many and less thick lifts or layers does not necessarily result in higher unit weights or lower conductivity values. As can be seen from the above overall averages of the unit weights, the unit weights did vary by a significant amount in all the three test strips. Furthermore, Figs. 53 and 54 shows that placing the clay in fewer lifts might actually produce a more uniform soil profile which could result in lower conductivity. Also, fewer lifts will means fewer interfaces and, therefore, lower lateral flow of water of leachate.

Field desiccation crack study

The constructed test strips were also used to study cracking due to the desiccation of the compacted clays. This was performed by marking two locations within each test strip immediately after the construction of the test strips. The locations and areas of these study locations were selected arbitrarily and are shown in Fig. 51. Within each study location the formation of cracks with time and temperature were noticed, and approximate crack widths were also recorded. After a period of 5 days at an average temperature of 90°F the depth of selected cracks at each location was approximately measured. This was done by injecting food coloring through the cracks and, then, tracing the depth of penetration of the coloring. A period of medium

intensity of rain was experienced during the first 2 days of the desiccation study. To prevent rain water from interfering with the study, the entire test strip areas were covered with a Visqueen sheet (plastic sheet). It was thought that the practice would prevent cracking of the underlying clays.

Desiccation cracks appeared 2 to 3 hours after the completion of the test strips. These cracks had a width of up to 2 millimeters (mm). After the rainy period was over, the Visqueen sheet was uncovered and many small cracks were observed. This indicates that the Visqueen sheet did not stop desiccation cracks but rather minimized them. The crack study locations generally showed that there are two types of cracks, main and minor cracks, as can be seen in Fig. 55. This could be a function of the size of clay clods and the presence of clay zones with relatively higher plastic activity than adjacent zones. This, in turn, led to desiccation activity to penetrate to a greater depth and by a larger amount. The range of widths and depths together with the corresponding average values are also shown in Fig. 55. However, below a depth of about 20 to 70 mm, all studied cracks had a width of less than 1 mm. This indicated that most of clay desiccation activities took place within the top 30 to 40 mm.

Field infiltration tests

A total five field infiltration tests were performed, the first three on test strip number 1 (3 lifts) and the last two tests performed on test strip number 2 (one lift). The locations of these infiltration tests are shown in Fig. 51. The sampler for the first infiltration test was pushed in the compacted clays by hand jacking while the samplers for all the other tests were pushed in by a backhoe. Infiltration tests were set up following procedures discussed earlier.

After the field infiltration tests were set up, readings of the water levels in the small plastic tube were taken at 2 to 4.5 hours and at 3 to 8 days. Then coefficients of infiltration were calculated using equation 11. The calculated values are shown in Table 7 together with the corresponding dry unit weights and the moisture contents.

Laboratory tests

Since the dry unit weight and the moisture content for the three test strips were determined earlier, no additional measurements of these two properties were obtained. However, the average unit weight and moisture content used in subsequent calculations are shown in Table 7.

Due to sample disturbance during extruding, only one laboratory hydraulic conductivity was performed. This was done on the sample obtained from the first infiltration test.

The test was performed using a hydraulic gradient of 347, and the predicted conductivity value was $5.4 * 10^{-9}$ cm/s.

Conductivity using suction and infiltration values

The above dry unit weight and moisture content were used in Fig. 33, and suction values (H_s) for all clay samples were obtained. Then, equation 34 was used to predict the saturated conductivity values. The predicted values are shown in Table 7.

Comparison between the predicted conductivity values

All the obtained laboratory and field test results are shown in Table 7. The formulas used to calculate these values are the same as those shown in Table 6. Laboratory conductivity test results that were measured by Ardaman and Associates and Jammal and Associates, together with the corresponding unit weights and moisture contents, are also shown in Table 7. Since the values of the unit weight and moisture content of the last three field infiltration tests were outside the extent of Fig. 33, an accurate prediction of the H_s vales for those samples could not be determined and, therefore, will not be included in the comparison.

As can be seen from the conductivity values in Table 7, the predicted conductivity values based on the suction and field infiltration test results compare very closely with

those obtained in the laboratory by the author and the other two companies.

Western Evaporation Basin-Bottom Liner

General

The western evaporation basin is located southwest of the main liner and was constructed under the same specifications as the main clay liner. The western and eastern evaporation basins were constructed before the main liner. Figure 50 shows the locations of the two basins together with the approximate locations of the performed field infiltration tests. Seven days after the beginning of the field tests a relatively long period of heavy rains which lasted about 6 weeks was experienced. Consequently, field readings were taken only for the first 6 days.

Field infiltration tests

A total of three field infiltration tests were performed at the general location shown in Fig. 50. The samplers for all three infiltration tests were pushed into the compacted clays by a backhoe. Infiltration tests were set up following procedures discussed earlier in this chapter.

After the field infiltration tests were set up, readings of the water levels in the small plastic tube were taken at 1.5 to 15 hours and at 5 to 6 days. Then the coefficient of infiltration was calculated using equation 11. The

calculated values are shown in Table 8, together with the corresponding dry unit weights and the moisture contents.

Laboratory tests

In the laboratory the dry unit weight and the moisture content tests were performed on the clay samples that were tested for field infiltrations. The measured dry unit weight and moisture content for the three field-obtained samples are shown in Table 8. Table 8 also shows a number of unit weight and moisture content that were measured by both Ardaman and Associates and Jammal and Associates. The measured values by the author correlate very well with those obtained by the aforementioned two companies.

One very important result is that the unit weight measured by the sampler with an angled cutting shoe (1SH and 2LA) was lower than that measured using the sampler with the straight cutting shoe (3LS). This fact proved that soil disturbance due to the cutting shoe angle is negligible. Furthermore, the variations that were observed in the measured unit weight are due to inhomogeneity in the in-situ soil unit weight.

Three laboratory hydraulic conductivity tests were performed on the field-obtained clay samples which were used to test for field infiltration tests. The laboratory conductivity tests were performed under a relatively high hydraulic gradient. This was necessary due to the high unit

weight of the tested soils and lack of time. The predicted values are shown in Table 8 and found to range from 3.43 * 10^{-11} to 8.36 * 10^{-10} cm/s.

The relationship between hydraulic conductivity and hydraulic gradient for the field-obtained sample (2LA) was obtained. The result is shown in Fig. 56. Figure 56 shows that as the gradient increases so does the conductivity values up to a maximum gradient of 180 which is defining the maximum past preconsolidation pressure. Beyond this value an increase in hydraulic gradient will not produce a significant change in the value of conductivity. This behavior is similar to those obtained in Chapter 2.

Conductivity using suction and infiltration values

The above dry unit weight and moisture content were used in Fig. 33, and suction values (H_s) for all clay samples were obtained. Then, equation 34 together with the information obtained from the field infiltration tests were used to predict the saturated conductivity values. The predicted values are shown in Table 8.

Comparison between the predicted conductivity values

All the obtained laboratory and field test results are shown in Table 8. The formulas used to calculate these values are the same as those shown in Table 6. Laboratory conductivity test results that were measured by Ardaman and

Associates and Jammal and Associates together with the corresponding unit weights and moisture contents are also shown in Table 8. The average values are in parentheses.

As can be seen from the conductivity values in Table 8, the predicted conductivity values based on the suction and field infiltration test results compare very closely with those obtained in the laboratory by the Jammal and Associates and Ardaman and Associates. The laboratory values that were measured by the author were higher by up to one order of magnitude than the rest of the measured values. This is due to the fact the author used a higher hydraulic gradient which leads to lower conductivity values.

TABLE 6. Comparison of Conductivity Values Obtained by Different Methods (S.W. Alachua Landfill).

Field Infiltr. Test No./Type	After 4.5 Hours (a)	After 5 Days (b)	Laboratory Test Results (c)	Suction + (a) (d)	Ardaman and Assoc. Lab Test Results (e)
1 SH	$6*10^{-7}$ cm/s	$1.96*10^{-8}$ cm/s	---	$3.04*10^{-9}$ cm/s	$4.7*10^{-9}$ cm/s
2 LA	$4*10^{-7}$ cm/s	$1.0*10^{-7}$ cm/s	$7.7*10^{-8}$ cm/s*	$1.17*10^{-9}$ cm/s	$3.3*10^{-9}$ cm/s

*Stabilized reading after 4 weeks
SH = Short sleeve
LA = Long sleeve angled
(a) & (b): $K = (a * L)/(A * \%) \ln h_1/h_2$
(c): $K = (Q * L)/(A * T * H_o)$, where i = 70
(d): $K = (Q * L)/(A * T * (H_o + H_s)$, 1 SH: γ_d = 105 pcf and w = 24.6%
2 LA: γ_d = 106.8 pcf and w = 20.6%
(e): Value selected for 1 SH: γ_d = 107.3 pcf and w = 20.1%
2 LA: γ_d = 104.3 pcf and w = 23.2%
Ardaman Ranges: γ_d = 89.4-111.9 pcf, w = 17.7-29.8%, $K = 7.4*10^{-8} - 2.1 * 10^{-9}$ cm/s

TABLE 7. Comparison of Conductivity Values Obtained by Different Methods (Astatula Field Test Strips).

Test Strip/ No. Lifts	Field Infilt. Test No.	After X No. of Hours (a)	After X No. of Days (b)	Lab Value on Undis. Field Sample (c)	Suction + (a) (d)	From Ardaman and/or Jamal & Assoc. (e)
1/3	1 LA	X = 4 hrs $12*10^{-7}$ cm/s	X = 5 days $8*10^{-7}$ cm/s	$5.4*10^{-9}$ cm/s**	$6.96*10^{-9}$ cm/s	$7.8*10^{-9}$ cm/s
1/3	2 LA	X = 4 hrs $15*10^{-7}$ cm/s	X = 8 days $17*10^{-7}$ cm/s	Av. w = 42.4% γ_d = 95 pcf	$3.82*10^{-9}$ cm/s	$9.4*10^{-9}$ cm/s
1/3	3 LS	X = 4 hrs $29*10^{-7}$ cm/s	X = 5 days $9*10^{-7}$ cm/s	w = 34.8% γ_d = 77.9 pcf	$5.5*10^{-8}$ cm/s*	$4.4*10^{-9}$ cm/s
2/1	1 LA	X = 4.5 hrs $44*10^{-7}$ cm/s	X = 8 days $13*10^{-7}$ cm/s	w = 33.4% γ_d = 78.7 pcf	$5.97*10^{-8}$ cm/s*	$9.8*10^{-9}$ cm/s
2/1	2 LS	X = 2 hrs $323*10^{-7}$ cm/s	X = 3 days $69*10^{-7}$ cm/s	w = 33.4% γ_d = 78.7 pcf	$2*10^{-7}$ cm/s*	$7.4*10^{-9}$ cm/s

*Outside the extent of Fig. 33.
**Stabilized reading after 4 weeks
LA = Long sleeve angle
LS = Long sleeve straight
(c): 1 LA: γ_d = 77.9 pcf, w = 34.8%, i = 347
(e): 1 LA: γ_d = 99.1 pcf, w = 24.9%, i = 69
2 LA: γ_d = 100.6 pcf, w = 23.6%, i = 67.4
3 SL: γ_d = 101.5 pcf, w = 26.7%, i = 65.8
1 LA: γ_d = 101.9 pcf, w = 23.2%, i = 69.2
2 LS: γ_d = 107.7 pcf, w = 21.1%, i = 62.2

TABLE 8. Comparison of Conductivity Values Obtained by Different Methods (Astatula Western and Eastern Evaporation Basins).

Field Infilt. Test No./ Type	After X No. of Hours (a)	After X No. of Days (b)	Lab Value on Undis. Field Sample (c)	Suction + (a) (d)	Ardaman and Assoc. (e)	Jamal and Assoc. (f)
1 SH	X = 1.5 hrs $745*10^{-7}$ cm/s	X = 5 days $9*10^{-7}$ cm/s	$8.36*10^{-10}$ cm/s*	$4.19*10^{-9}$ cm/s	$1.1*10^{-9}$ cm/s East Basin	$3.2-9.4*10^{-9}$ cm/s (Av=5.5)
2 LA	X = 15 hrs $81*10^{-7}$ cm/s	X = 6 days $29*10^{-7}$ cm/s	$3.43*10^{-11}$ cm/s*	$7.87*10^{-9}$ cm/s	$7.9*10^{-10}$ cm/s West Basin	$4.1-9.8*10^{-9}$ cm/s (Av=6.5)
3 LS	X = 15 hrs $105*10^{-7}$ cm/s	X = 6 days $29*10^{-7}$ cm/s	$2.94*10^{-10}$ cm/s*	$7.87*10^{-9}$ cm/s	---	$2.4-7.8*10^{-9}$ cm/s (Av=5.8)

*Stabilized reading after 4 weeks
SH = Short sleeve
LA = Long sleeve angled
SL = Long sleeve straight
(c): 1 SH: γ_d = 99.2 pcf, w = 23.32%, i = 404, 7/26 to 8/4
2 LA: γ_d = 100.14 pcf, w = 21.6%, i = 302, 7/28 to 8/8
3 SL: γ_d = 103.0 pcf, w = 21.71%, i = 440, 8/4 to 8/21
(e): East Basin: γ_d = 94.1 pcf, w = 27.8%, i = 132-61
West Basin: γ_d = 111 pcf, w = 19.6, i = 132-61
(f): 1st Lift: γ_d = 100.6-107.8 (104.3) pcf, w = 21.9-24.1 (23.2)%, i = 67.4-109.4
2nd Lift: γ_d = 101.5-107 (104.3) pcf, w = 21-26.7 (23)%, i = 62.2-76.05
3rd Lift: γ_d = 99.1-107.7 (104.3) pcf, w = 21.1-24.9 (22.6)%, i = 62.2-81

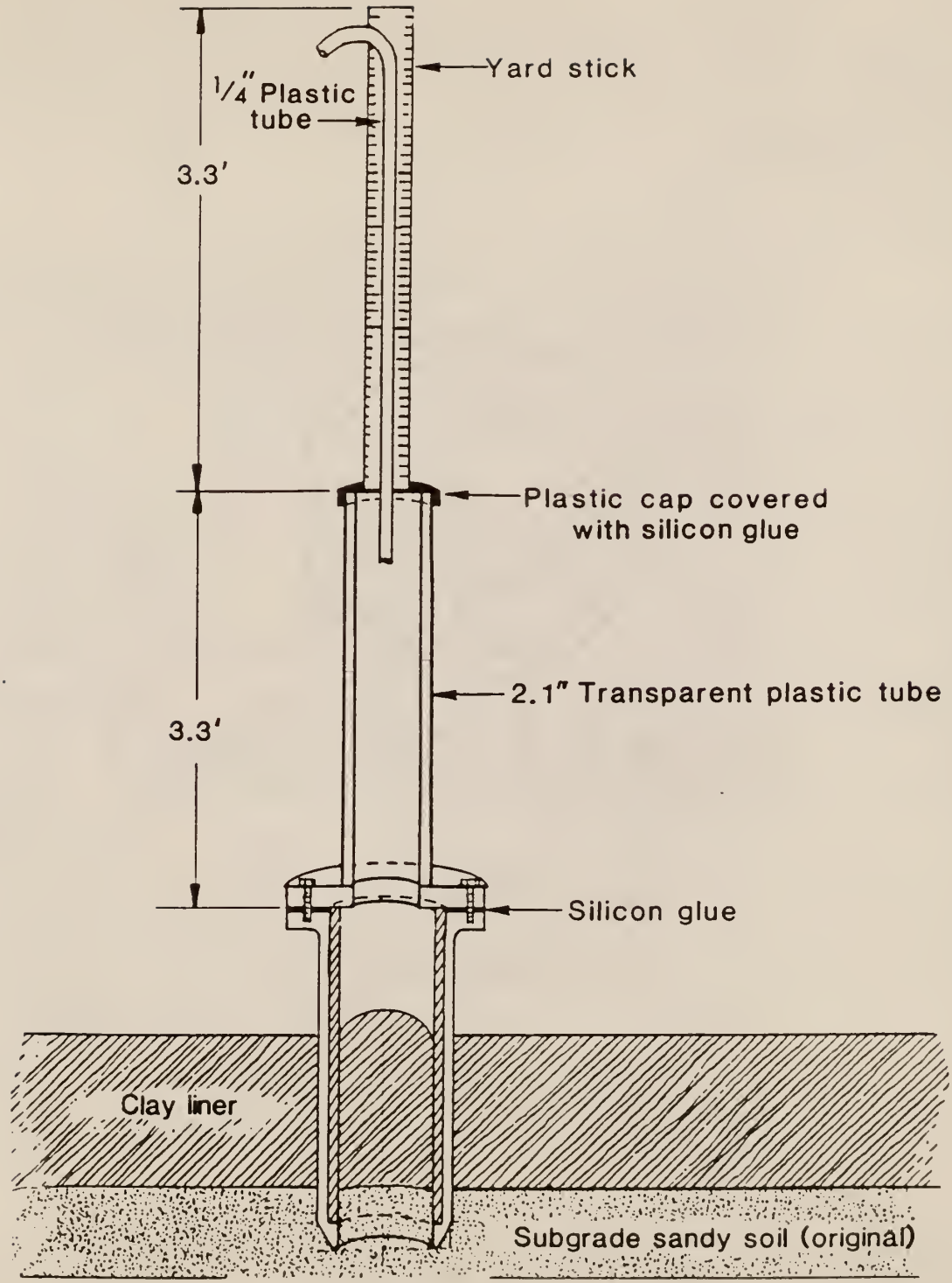


Fig. 45. Field Infiltration Test Setup.

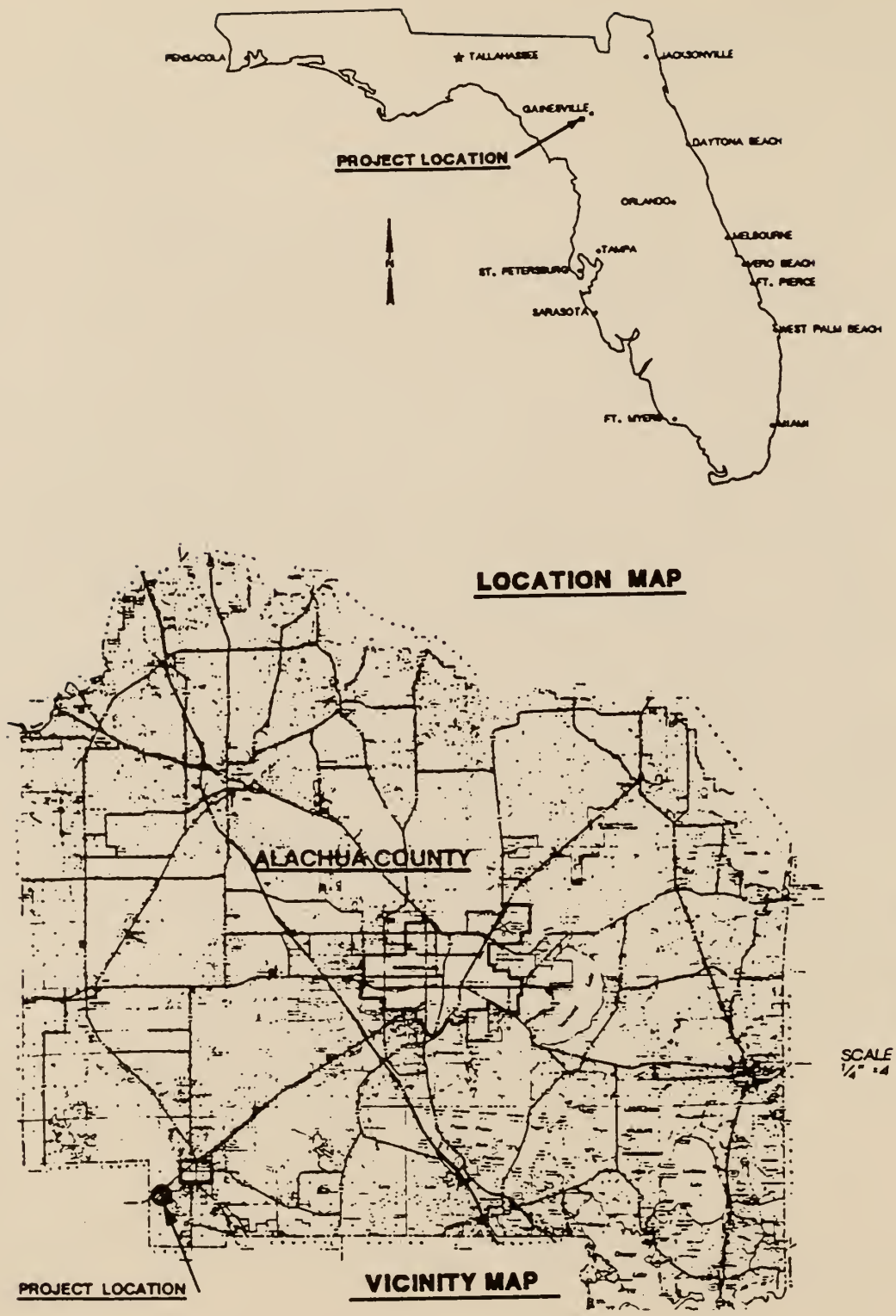


Fig. 46. Location and Vicinity Map of S.W. Alachua Landfill.

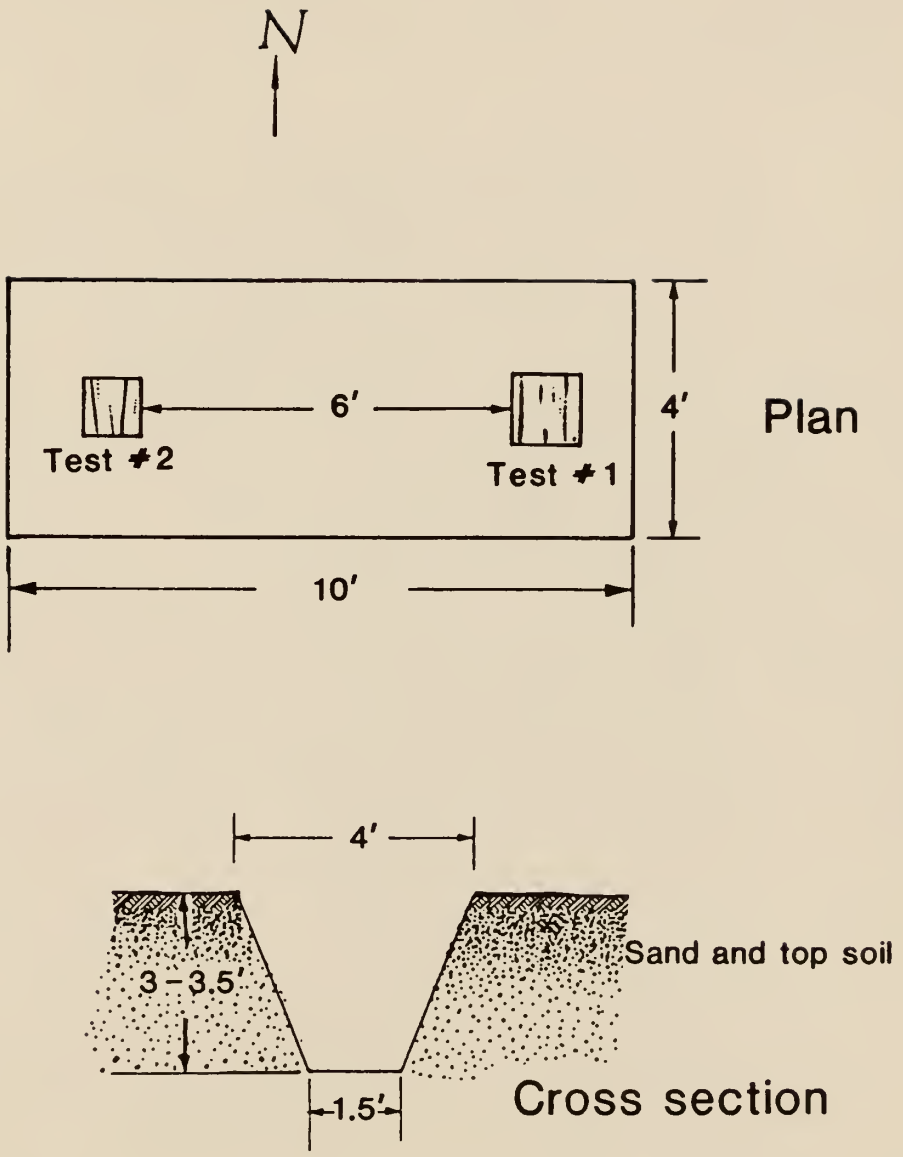


Fig. 47. Field Infiltration Test Locations and Cross Section (S.W. Alachua Landfill-Top Cover).

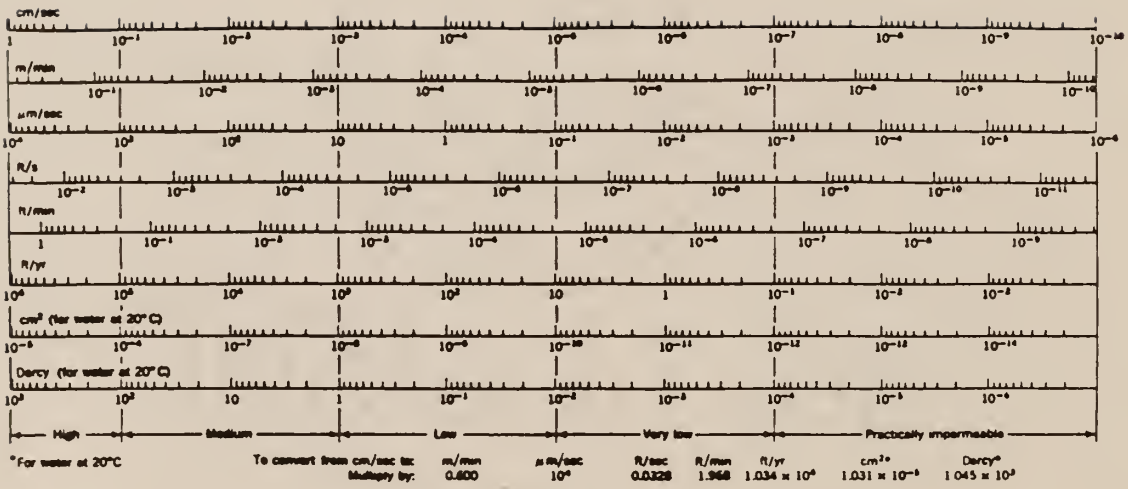


Fig. 48. Various Scales of Reporting Hydraulic Conductivity Values (Lambe and Whitman 1979).

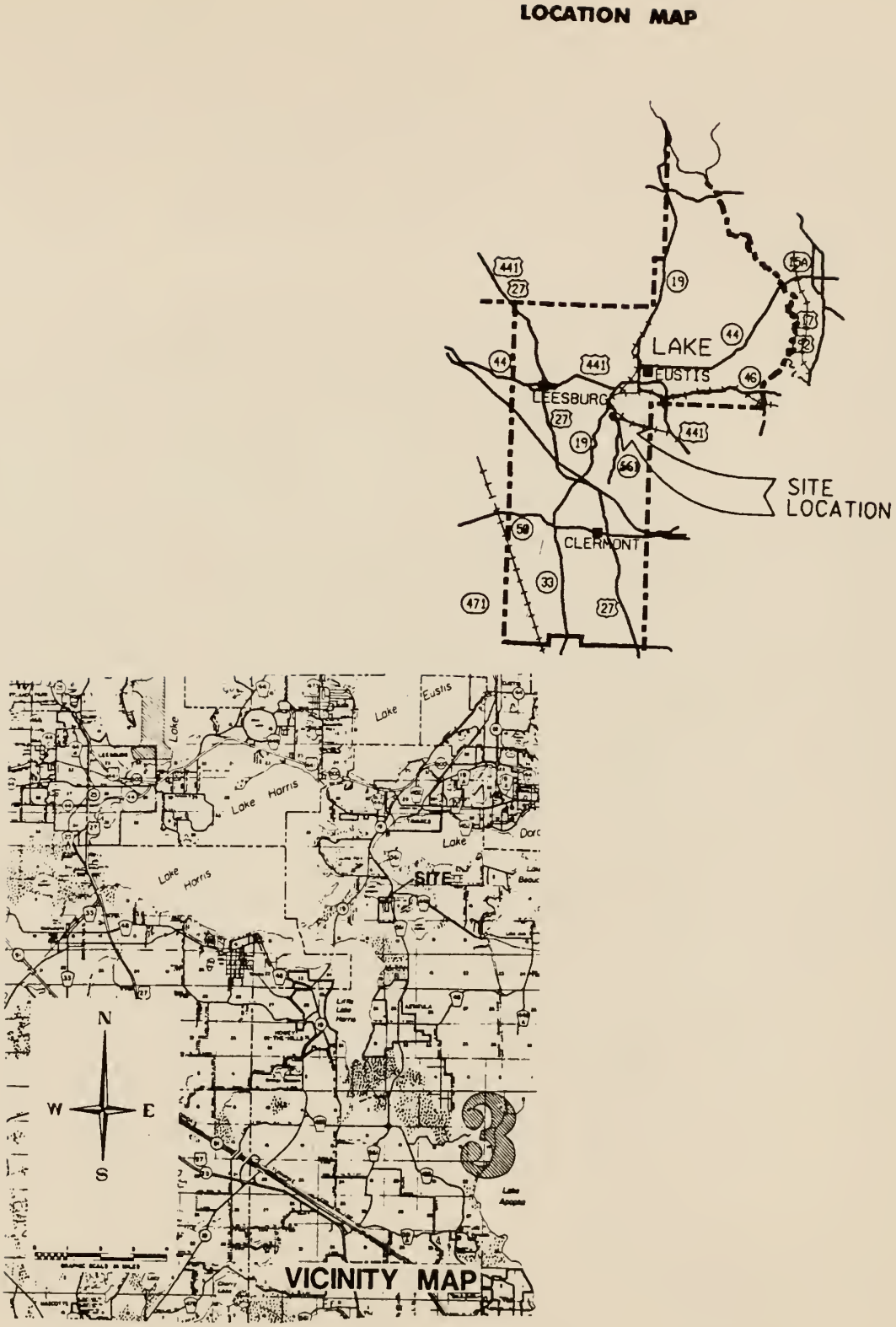


Fig. 49. Location and Vicinity Map of Astatula Ash Residue Monofill Landfill.

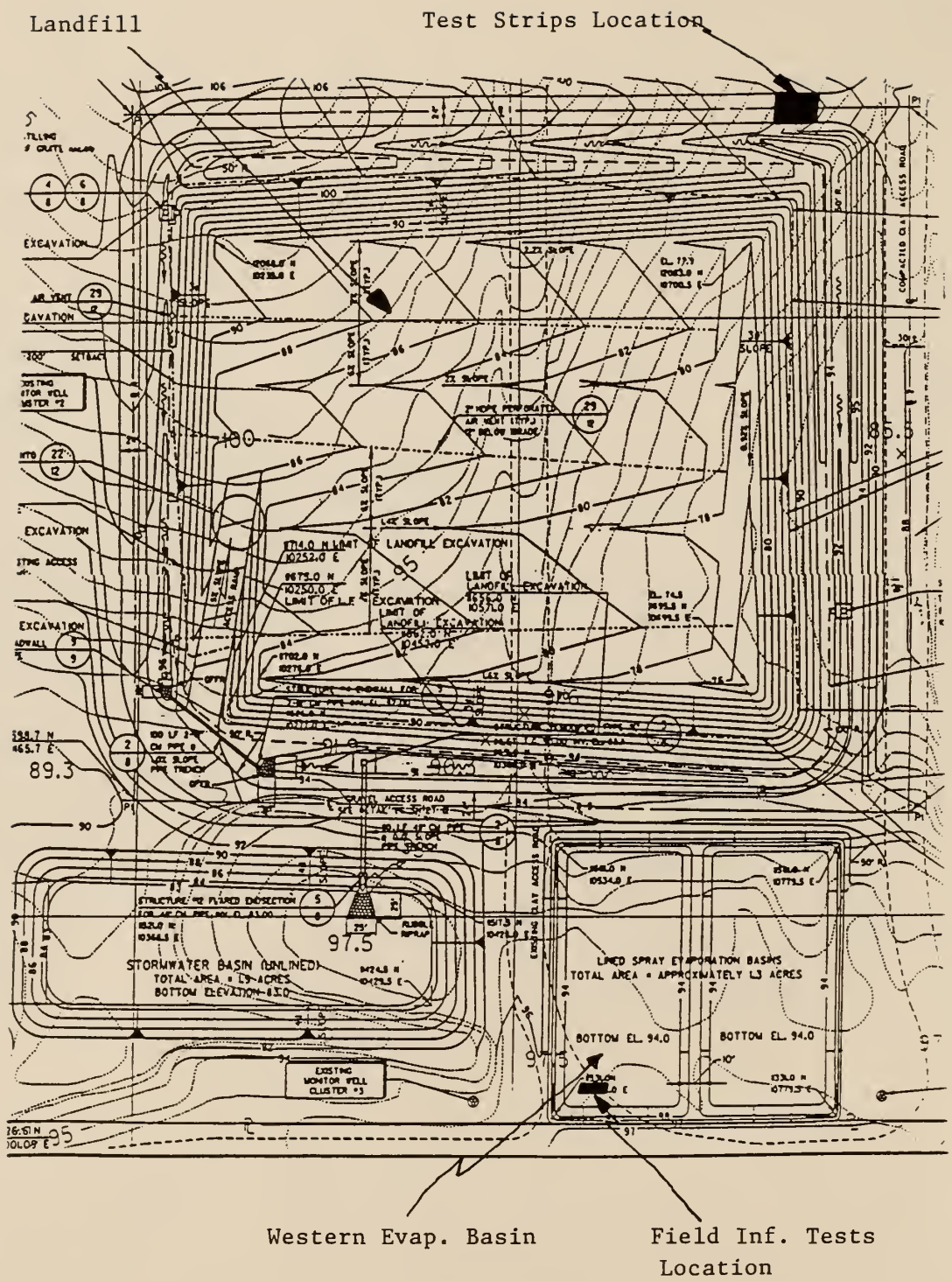


Fig. 50. General Location of Test Strips, Landfill, and Evaporation Basins.

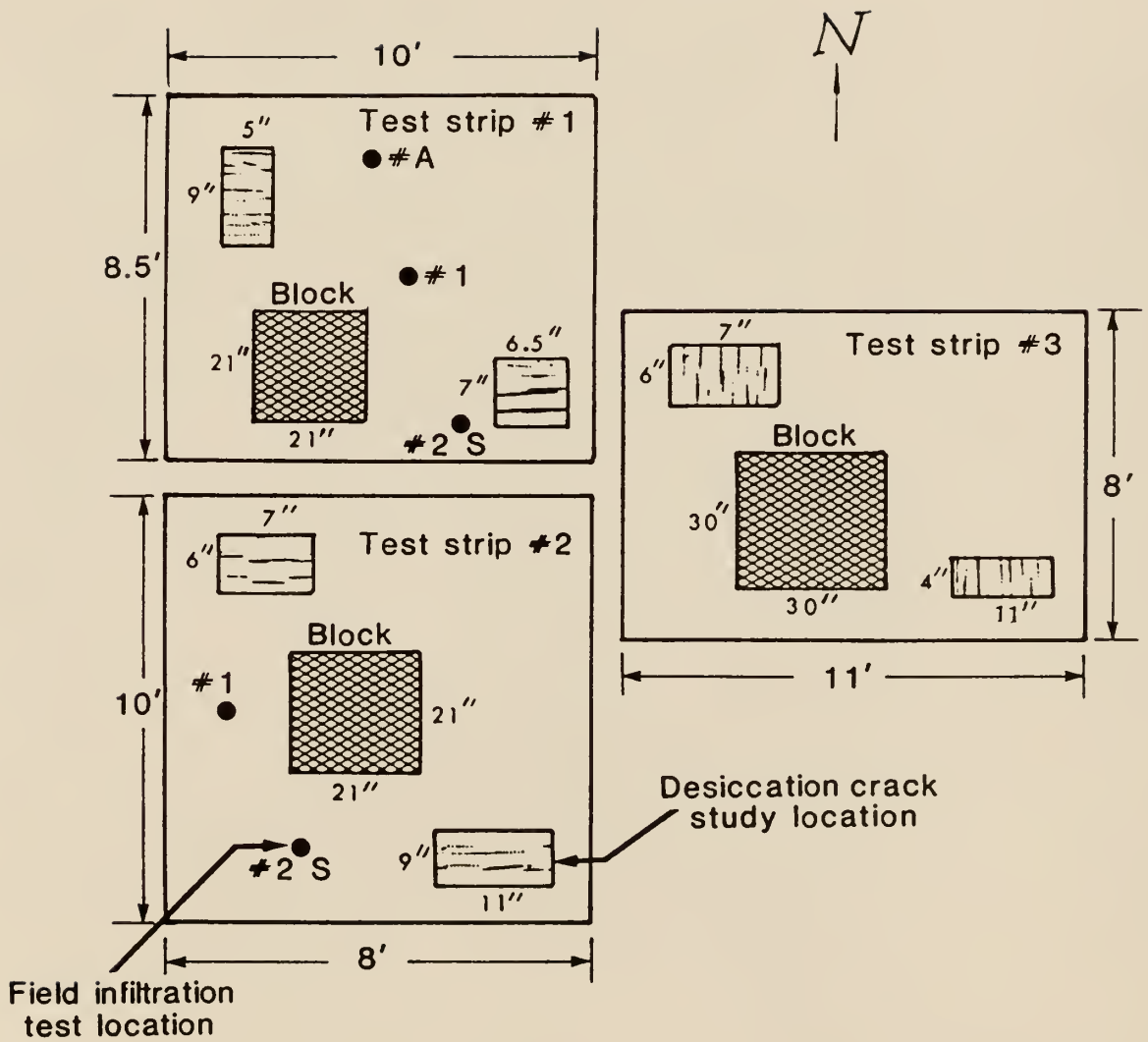


Fig. 51. Test Strips Showing Dimensions and Locations of All Performed Field Tests.

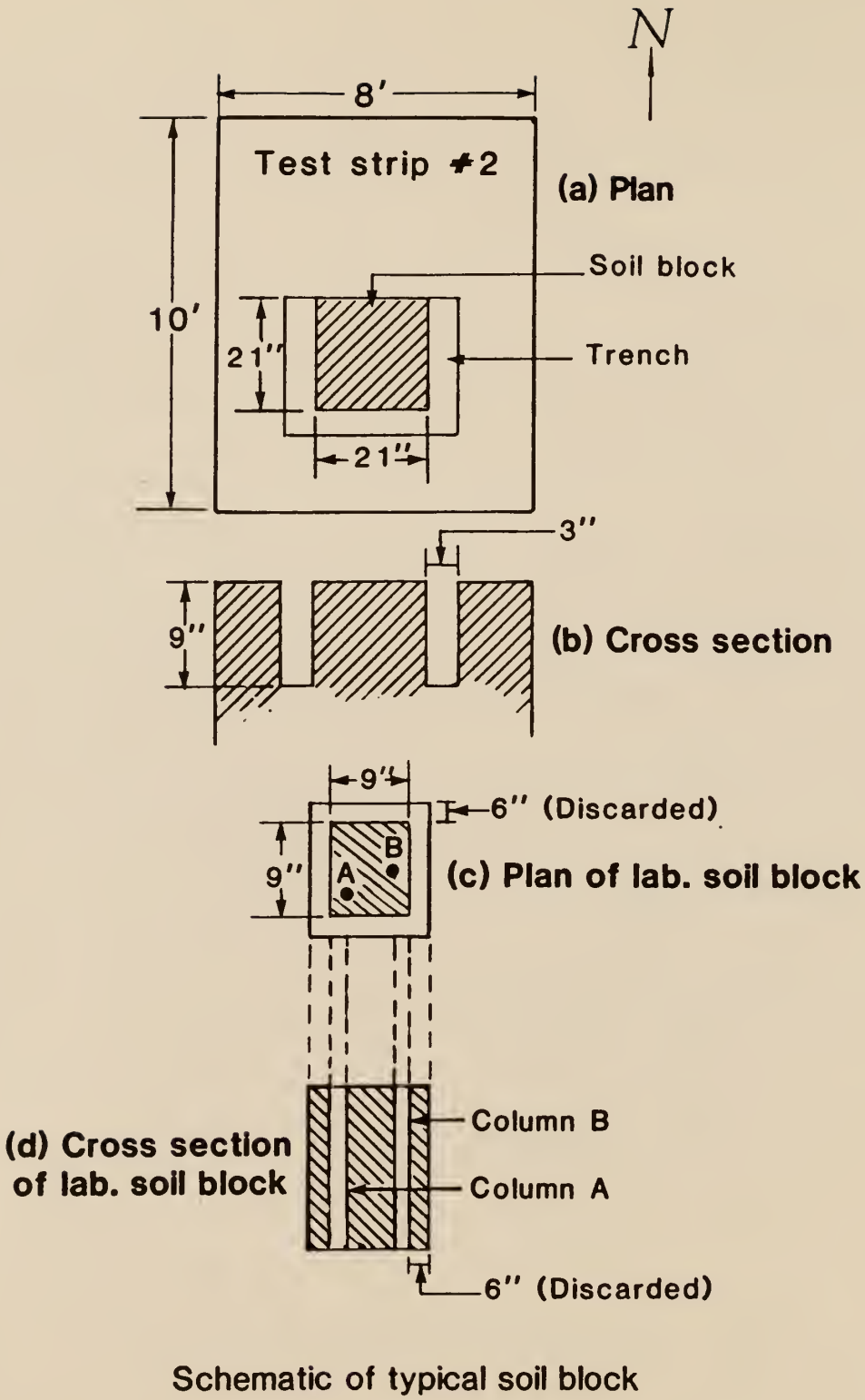


Fig. 52. Schematic of Typical Soil Block Showing All Dimensions.

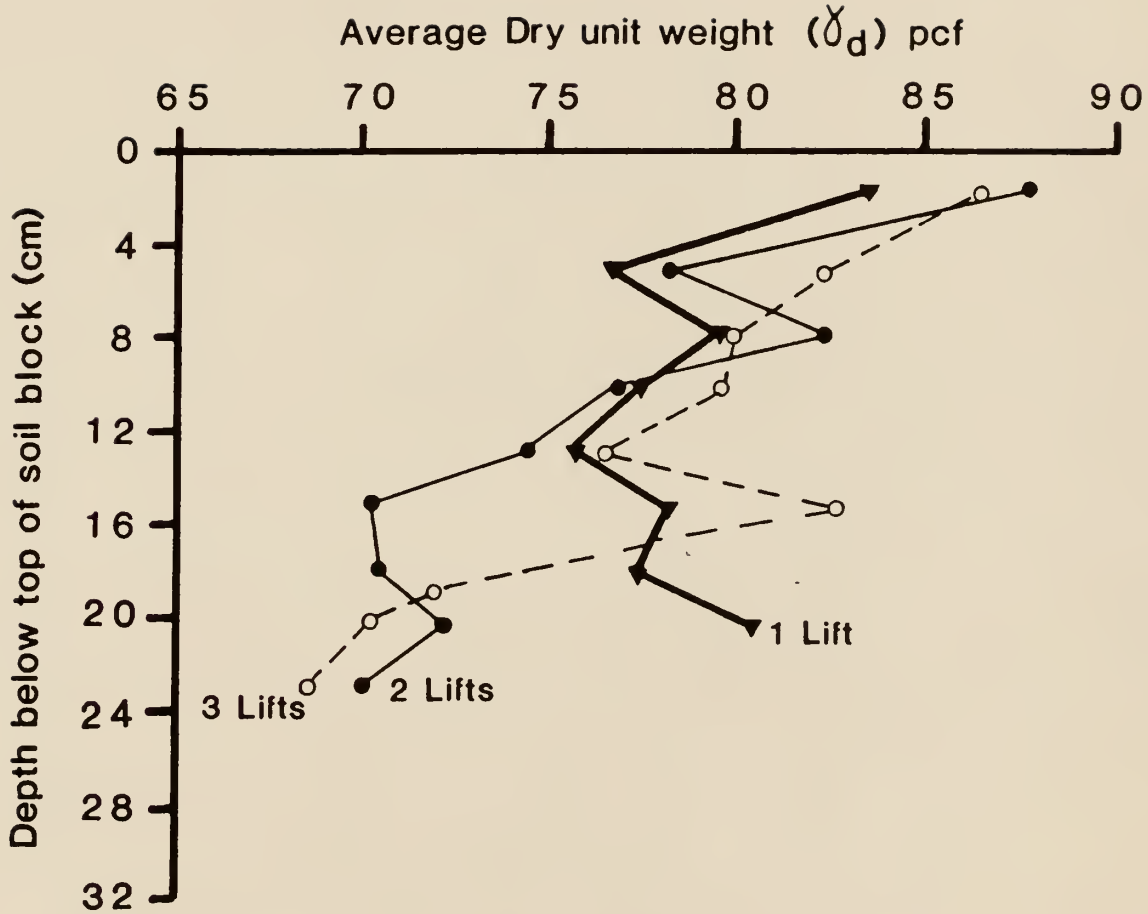


Fig. 53. Average Dry Unit Weight vs. Depth of Soil Block.

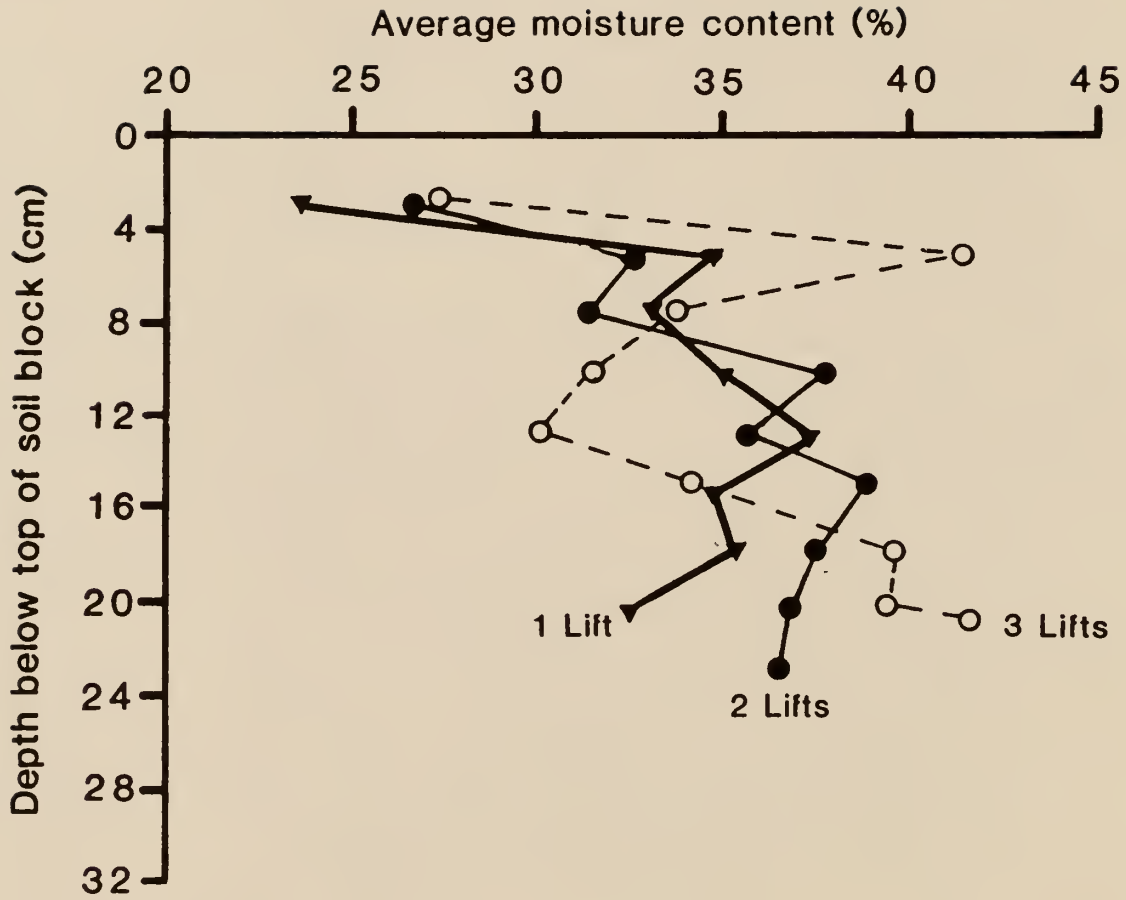


Fig. 54. Average Moisture Content vs. Depth of Soil Block.

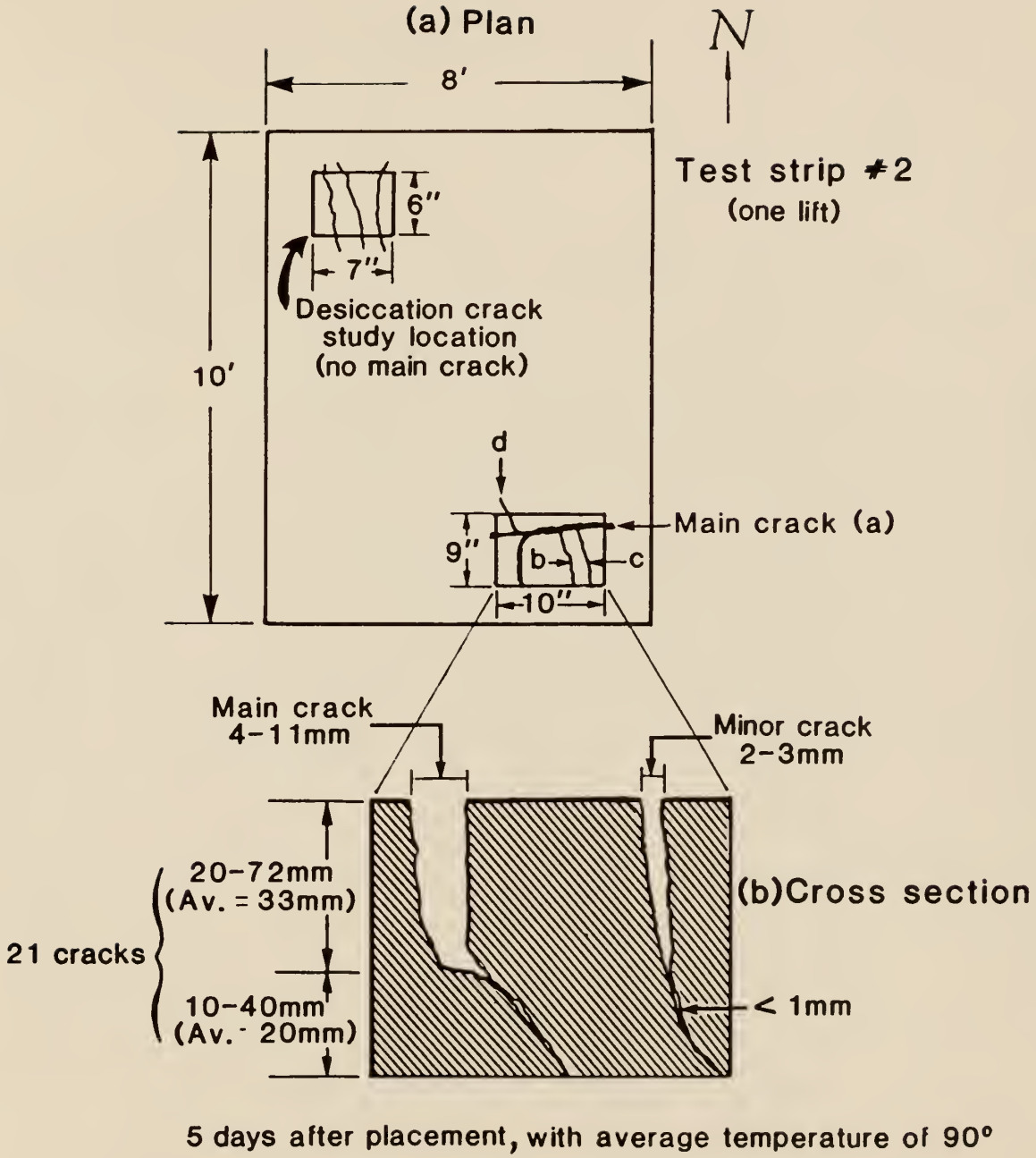


Fig. 55. Typical Desiccation Crack Study Location and Cross Section.

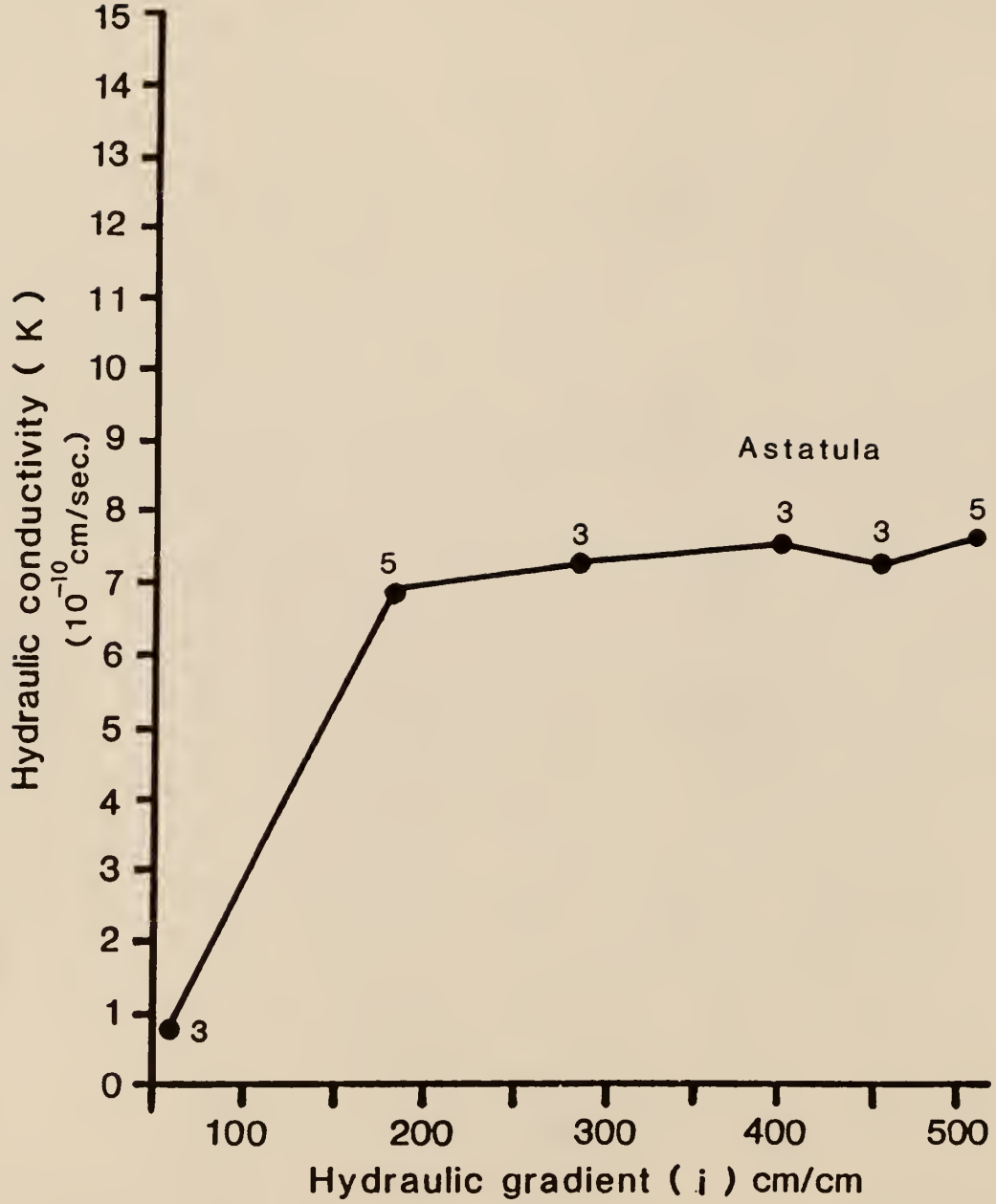


Fig. 56. Hydraulic Conductivity vs. Hydraulic Gradient on Field Obtained Sample (Astatula Western Evaporation Basin).

CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The conclusions reached by the author are numerous and can be divided into those obtained based on the literature surveyed, laboratory findings, and the field findings.

Conclusions Based on the Literature Surveyed

The literature review outlined in Chapter 1 showed that there is no set standard for performing laboratory hydraulic conductivity tests on soils with conductivities lower than 10^{-7} cm/s. Furthermore, currently there is no agreement on whether to include the effect of porosity and effective porosity in the calculations of laboratory conductivity.

The laboratory conductivity tests are currently performed on field-obtained samples that are relatively very small. Because of this and the fact that these samples are subjected to a very high degree of disturbance during sampling and sample preparation, the predicted conductivity value tends to be somewhat erratic and sometimes tends to overestimate the actual value.

There are no reliable, rapid, and reproducible methods for determining hydraulic conductivity or infiltration

coefficients in the field. This is due to the high value of suction in clayey soils. In addition, the current field methodology of performing the conductivity tests are much different than those performed in the laboratory, and yet the two results are compared frequently.

Hydraulic conductivity is becoming the most important soil property, and at the same time it is the most difficult property to measure because it is very sensitive to a great many factors.

Conclusions Based on Laboratory Findings

A combination of a field sampler (Figs. 30 and 45) and a simplified rigid wall permeameter (Fig. 24), similar to that used in this study, can be used to obtain undisturbed field samples and test for hydraulic conductivity of fine-grained soils. This combination will cause much less soil disturbance, facilitate testing a larger sample, and can be performed in a shorter period of time and, hence, is superior to the existing methods.

The illusive soil suction can be determined in the laboratory much more efficiently and quickly than using any other method. Contrary to previous researchers, soil suction is a function of both dry unit weight and the moisture content, as was proven in this study (Fig. 33). Soil suction increases with increasing dry unit weight and decreasing moisture content. Furthermore, the relationship between soil

suction, dry unit weight, and moisture content is similar to that of the degree of saturation (Fig. 31). Soil suction obtained by the method used in this research can be used together with some basic infiltration data that are obtained very rapidly in the field to predict very accurately the much desired field-saturated hydraulic conductivity value (Tables 6, 7, and 8).

The value of hydraulic conductivity is insensitive to the thickness of the soil layer. This is especially true if the soil is homogeneous and isotropic. The hydraulic conductivity value tends to appear to be lower for thicker soil samples. This is mainly due to the method of building the soil thickness and the nonuniform distribution of various mechanical and physical soil properties with increasing soil thickness.

Placing the soil in a larger number of lifts tends to lower the hydraulic conductivity by up to 100% (double the value). This is because as the number of soil layers increases, the unit weight increases, leading to a reduction in soil porosity that results in lower conductivity.

Increasing the applied hydraulic gradient tends to increase the value of conductivity, especially for soil with a low initial dry unit weight. Beyond the maximum past preconsolidation pressure of the soil, the soil hydraulic conductivity tends to be independent of the applied hydraulic gradient.

Under a constant hydraulic gradient the hydraulic conductivity tends to fluctuate with time up to a certain time, depending on type of soil and soil properties. Beyond this certain time the hydraulic conductivity tends to stabilize at a constant value. These changes of hydraulic conductivity are more pronounced in soil with low initial dry unit weight than highly dense soils.

The moisture content distribution is never uniform, and it is an error to assume at any stage that the soil is fully and completely saturated. This is shown by numerous test results.

Desiccated fine-grained soils tend to regain their usual low hydraulic conductivity values upon rewetting by swelling into the voids and closing the desiccation cracks that were created by drying. In fact, this behavior, which is also termed self-healing, is observed in the field more vividly.

Conclusions Based on Field Findings

Field samplers (Fig. 30) such as those designed and used in this study can be used to obtain field undisturbed soil samples, for laboratory conducted hydraulic conductivity tests, and used at the same time to perform a quick and rugged field infiltration tests (Fig. 45). It was shown that using these samplers causes minimum soil disturbances.

The field infiltration test setup (Fig. 45) that was used in this study is very rugged, rapid, not messy, does not

involve expensive or sensitive equipment, can be run by any engineering technician, and measured values that when used with laboratory suction tests (Fig. 33) resulted in hydraulic conductivities that were very close to the laboratory values (Tables 6, 7, and 8).

The proposed combination of field infiltration and laboratory-obtained suction can be used in the quality control and quality assurance program of liner construction. This method is definitely simpler, more economical, and quicker than existing methods.

Dry unit weight and moisture content distributions of compacted clay liners will vary with depth. The least variation was given by liners that were placed in a few layers than those placed in numerous layers. Furthermore, interfaces between layers will result in higher lateral flow of water.

Plastic soils do crack due to the desiccation process. Crack dimensions will vary and are highly dependent on many factors, most of which are related to the type of soil, soil conditions, and temperature. Clay soils can form desiccation cracks within a few hours. Average depth of cracks after 5 days of exposure was about 33 mm. Covering the surface of the exposed clay liner with visqueen will not prevent cracking but minimize it to a certain degree.

Recommendations

Recommendations for Laboratory Works

Future laboratory hydraulic conductivity tests should be designed such that they could be performed directly on the field-obtained undisturbed samples with minimum amount of preparation. A larger sample should be tested instead of the current sizes of 1 to 2 inches in diameter and up to 3 inches in length.

A set methodology should be established on which hydraulic conductivity testing and calculations of the conductivity value are based.

Rigid wall type of hydraulic conductivity testing should be developed further as it has good potential.

Recommendations for Field Work

Field infiltration and hydraulic conductivity tests should be studied much more as the geoenvironmental field becomes a more important part of geotechnical/civil engineering.

Field testing should be geared more in the direction of obtaining an infiltration coefficient rather than the saturated hydraulic conductivity, since it is very difficult to saturate the tested soils in the field.

Field test strips should be constructed at the beginning of any landfill project. These test strips then should be

used to perform various field tests such those performed by this study.

Recommendations for Future Research

The areas that require research in clay liners and clay conductivity are numerous. This is because clay liner technology is moderately understood, and clay conductivity is the least understood and studied property in geotechnical engineering.

Further research should be performed in developing a rugged and rapid field method for measuring the infiltration coefficient.

The effect of sample diameter and thickness on the conductivity should be investigated. This investigation can be carried out in the field and laboratory.

Variations of soil suction with the type of filter paper that contain different pore sizes and structure to that of tested clay should be studied further.

Effect of aging on the conductivity value should be studied further. Virtually no information is available on this subject.

The viability of utilizing clay soils to line underground gasoline and other hazardous buried tanks should be studied, as no guidance is available on this subject.

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APPENDIX A
PHYSICAL AND INDEX PROPERTIES OF THE RESEARCH CLAY (OBTAINED BY THE AUTHOR)

1. Visual Engineering Description of Soil (ASTM D2488):

   Light Gray and Green Mottled Reddish and Yellowish Brown Silty Clay with Trace to Some Fine to Medium Subangular Sand, and Occasional fine to Coarse Gravel Size Limestone Nodules.

2. Natural Moisture Content, As Received, (ASTM D2216):

   Sample No. 1 = 20.5 percent

   Sample No. 2 = 14.6 percent

   Sample No. 3 = 21.5 percent

   Sample No. 4 = 18.3 percent

   Sample No. 5 = 15.4 percent

   Average Moisture Content = 18.1 percent.

3. Amount of Soil Finer Than the No. 200 Sieve (ASTM D1140):

   Sample No. 1 = 38.6 percent

   Sample No. 3 = 70.2 percent

Sample No. 5 = 40.3 percent

Average Amount Finer = 49.7 percent.

4. Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI) of Soils Finer Than the No. 40 Sieve, Atterberg Limits (ASTM 4318):

Sample No. 1:

* LL = 52, 54, 47, 49, and 45 percent.
  Average LL = 49 percent.
* Average PL = 18 percent.
* PI = 38 percent.
* ASTM (USCS) Fine Classification = CL-CH.

Sample No. 3:

* LL = 49, 49, 49, 50, and 49 percent.
  Average LL = 49 percent.
* Average PL = 17 percent.
* PI = 32 percent.
* ASTM (USCS) Fine Classification = CL-CH.

Sample No. 5:

* LL = 47, 49, 49, 50, and 52 percent.
  Average LL = 49 percent.
* Average PL = 18 percent.

* PI = 31 percent.

* ASTM (USCS) Fine Classification = CL-CH.

5. Classification of Soils for Engineering Purposes, Based on Unified Soil Classification System "USCS," (ASTM D2487):

Sample No. 1: SC, Clayey Sand.

Sample No. 3: CL-CH, Lean-Fat Sandy Clay.

Sample No. 5: SC, Clayey Sand.

6. Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI) of Soils Finer Than the No. 200 Sieve, Atterberg Limits, (ASTM D4318):

Sample No. 1:

* LL = 101, 102, 104, 101, and 101 percent.
  Average LL = 102 percent.
* Average PL = 28 percent.
* PI = 74 percent.
* ASTM (USCS) Fine Classification = MH.

Sample No. 3:

* LL = 100, 90, 86, 87, and 76 percent.
  Average LL = 88 percent.
* Average PL = 30 percent.
* PI = 58 percent.

* ASTM (USCS) Fine Classification = MH.

Sample No. 5:

* LL = 110, 105, 96, 95, and 90 percent.
  Average LL = 99 percent.
* Average PL = 31 percent.
* PI = 68 percent.
* ASTM (USCS) Fine Classification = MH.

7. Specific Gravity of Soils, Gs, (ASTM D854):

Sample No. 1: Gs = 2.64

Sample No. 3: Gs = 2.48

Sample No. 5: Gs = 2.54

Average Value of Gs = 2.55

APPENDIX B
MINERAL AND CHEMICAL PROPERTIES OF THE RESEARCH CLAY
(OBTAINED WITH PERMISSION OF MFM AND LAKE COUNTY AUTHORITY)

Mineral Properties

Sample Description	F.S.	Gradation Silt	Clay	Clay Minerals
No. 1 six inches below top of pre-mix	62.9	6.7	31.2	Montmorillonite
No. 2 pre-mix	63.2	4.7	32.1	Montmorillonite
No. 3 pre-mix with white veinlits	62.8	7.3	30.5	Montmorillonite
No. 4 green clay	19.1	17.2	63.0	Montmorillonite, Sepiolite, Attapulgite
No. 5 clayey sand above pre-mix	76.9	10.1	13.0	Kaolinite-weathered Illite-waverlite
No. 6 Fuller Earth	54.5	10.9	41.1	Montmorillonite, Sepiolite, Attapulgite
No. 7 Sand layer 5 feet above Fuller Earth	65.2	5.5	29.3	Montmorillonite, Sepiolite, Attapulgite
No. 8 Fuller Earth, 1 foot above sand lense	25.1	18.2	56.7	Montmorillonite, Sepiolite, Trace of Attapulgite
No. 9 Sand Layer below Fuller Earth	77.6	5.8	19.6	Montmorillonite, Sepiolite, Trace of Attapulgite
No.10 green clay - South end of deposit to trees	25.7	1.6	72.6	Montmorillonite

Samples Submitted by Dr. E.C. Pirkle for MFM
Charlotte County Project

Sample Code	MFM Sample No	Depth in Feet	Gradation - % FS	Silt	Clay	Clay Mineralogy
W10749	10	40 - 42	12.7	75.6	11.7	M^{++} S* Ca
W10750	7	36 - 40'	7.3	80.7	12.0	K^{+} M* Dol
W10750	8	40 - 42'	34.5	41.5	24.0	M^{++} S* Ca
W10750	9	42 - 46.5'	31.5	28.0	40.5	M^{++} K* S* Ca*
W10752	5	31 - 32.5	32.0	?	?①	M^{++} S* Ca Dol
W10752	6	34 - 38	12.3	19.2	68.5	M^{++} K^{+} S
W10752	1	36.5- 40	23.2	25.5	51.3	M^{++} I^{+} C^{+}
W10752	2	42. - 43.5	15.5.	2.6	81.9	M^{++}
W10752	3	51. - 55.	9.1	5.3	85.6	M^{++} S* A*
W10752	4	62. - 63.5'	3.4	9.7	86.9	M^{++} S* A*
W10760	14	34 - 38'	20.5	58.8	20.7	M^{++} S* Ca
W10760	16	54 - 57'	4.1	6.0	89.9	M^{++} S*
W10760	15	58 - 61	7.2	5.8	87.0	M^{++} A*
W10760	13	72 - 75	20.3	35.2	44.5	M^{++} S^{+} A* Ca*
W10760	12	86 - 90	67.7	7.4	24.9	M^{++} A*
W10479	11	45.5- 48.5	46.9	26.6	26.5	m^{++}- Mixed Layer - Dol

① Samples gelled. Could not determine percent clay.
Clays: M - Montmorillonite, I - Illite, S - Sepiolite, A - Attapulgite
++ - Major, + - Minor, * - Trace, Ca - Ca Co$_3$, Dol - Dolomite

Mechanical Analyses of Sand from Premix at Lowell

Sample No	Percent Sand[1] Retained on Mesh										Heavy Minerals in %
	14	20	35	45	60	80	120	170	230	pan	
1	0.62	0.73	8.09	20.12	33.71	14.73	7.47	10.06	4.36	0.10	0.31
2	2.40	1.05	7.48	18.31	31.83	16.68	8.44	9.78	3.93	0.10	0.32
3	3.57	1.09	7.85	19.07	32.87	14.60	7.85	9.14	3.87	0.10	0.34

[1]Quartz sand and heavy mineral sand

Percentages of Heavy Minerals from Premix at Lowell

Spl. No.	Heavy Minerals in %	Zircon	Ilmenite	Leucoxene	Rutile	Staurolite	Tourmaline	Spinel	Monazite	Kyanite-Sillimanite	Garnet	Epidote	Hornblende	Hematite	Others and Quartz
1	0.31	12.83	34.16	2.88	7.17	18.48	11.25	0.12	0.42	8.21	0.18	0.00	0.00	2.08	2.22
2	0.32	10.05	32.43	1.84	7.49	18.61	11.14	0.00	0.76	7.86	0.21	0.00	0.00	4.68	4.92
3	0.34	12.19	32.46	4.07	5.99	18.41	11.36	0.00	1.23	8.13	0.67	0.10	0.00	3.76	1.63

Table 1 - Comparison of Clay Liner Metals Content and Regulatory Criteria

TOTAL METALS

Metal Parameter (1)	USEPA Regulatory (3) Preferred Limits on Residue Application	FDER Regulatory (4) Preferred Limits on Residue Application	Typical (5) Levels In Soils	On-Site Sand	On-Site Red Clayey Sands		MFM Liner Clay Borrow		
					No. 1	No. 2	No. 1	No. 2	No. 3
Arsenic, mg/kg (2)	70	-	6	<5 (6)	<5	7.41	<5	<5	<5
Barium, mg/kg	-	-	10	<15	<15	<15	<15	15.0000	48.5000
Cadium, mg/kg	90	30	0.06	<1	<1	1.050	2.270	1.330	<1
Chromium, mg/kg	2700	-	100	<4	18.870	28.820	99.600	113.000	50.900
Lead, mg/kg	600	1000	10	<5	<5	<5	<5	<5	<5
Mercury, mg/kg	199	-	-	<0.02	<0.02	0.0260	<0.02	<0.02	<0.02
Selenium, mg/kg	810	-	0.5	<1	<1	<1	<1	<1	<1
Silver, mg/kg	-	-	-	1.1280	<1	<1	<1	<1	1.1400
Sodium, mg/kg	-	-	-	10.140	12.970	11.810	502.000	426.000	171.000

TCLP TEST(7)

Metal Parameter (1)	Regulatory Level (9)	MFM Liner Clay Borrow			Test Results
		No. 1	No. 2	No. 3	
Arsenic, mg/l (8)	5.0	<0.010	<0.010	<0.010	passed
Barium, mg/l	100.0	0.240	0.270	0.220	passed
Cadium, mg/l	1.0	0.005	0.005	<0.005	passed
Chromium, mg/l	5.0	0.015	0.017	0.013	passed
Lead, mg/l	5.0	<0.010	<0.010	<0.010	passed
Mercury, mg/l	0.2	<0.0005	<0.0005	0.0119	passed
Selenium, mg/l	1.0	<0.005	<0.005	<0.005	passed
Silver, mg/l	5.0	<0.010	<0.010	<0.010	passed
Sodium, mg/l	-	25.0	22.0	15.0	-

(1) Primary Drinking Water Parameter Metals.

(2) mg/kg = milligrams per kilograms, dry weight basis unless otherwise noted.

(3) From "Standards for the Disposal of Sewage Sludge; Proposed Rule", 40 CFR Parts 257 and 503, Federal Register, February 6, 1989, Table B-2, p. 5901.

(4) From FDER Chapter 17-640 FAC for the management and disposal of domestic wastewater residuals.

(5) Typical metal consentrations in soils from "Sludge Treatment and Disposal", Vol. 2, EPA-625/4-78-012.

(6) Less than (<) values mean less than detectable limits.

(7) TCLP - Toxicity Characteristics Leaching Procedure.

(8) mg/l = milligram per liter.

(9) USEPA regulatory levels for TCLP promulgated in the Federal Register on March 29, 1990.

POST, BUCKLEY, SCHUH & JERNIGAN, INC.
6631 E. COLONIAL DR
ORLANDO, FLORIDA 32807

age 1 PBS&J ENV LABS REPORT Work Order # 90-05-163
eceived: 05/18/90 06/12/90 15:19:45

REPORT TO: LAKE COUNTY LANDFILL-ASTATULA
PBS&J ORLANDO
800 N. MAGNOLIA AVE SUITE 600
ORLANDO, FL. 32803
ATTEN OMAR SMITH

CLIENT LAKE COUNTY SAMPLES 3
COMPANY LAKE COUNTY LANDFILL-ASTATULA
ACILITY

WORK ID CLAY LINER SAMPLES
TAKEN PROJECT #07-568.00
TRANS SOIL SAMPLES
TYPE
P.O. #
INVOICE under separate cover

PREPARED BY: PBS&J Environmental Lab.
6635 East Colonial Drive
Orlando, Florida 32807
DHRS/DER# 83170, E83011
ATTEN Kimberly Kunihiro
PHONE (407) 277-4443 AIHA # 213

Kimberly Kunihiro
CERTIFIED BY

CONTACT KUNIHIRO

We are pleased to provide this report of analysis. If you have any questions regarding this report or further analysis please feel free to telephone.

Previously Reported on 05/29/90.

SAMPLE IDENTIFICATION

1	CLAY SAMPLE 1
2	CLAY SAMPLE 2
3	CLAY SAMPLE 3

TEST CODES and NAMES used on this report

AG F	SILVER-FURNACE METHOD
AS F	ARSENIC-FURNACE METHOD
BA I	BARIUM-ICP METHOD
CD I	CADMIUM-ICP METHOD
CR I	CHROMIUM-ICP METHOD
HG	MERCURY
NA I	SODIUM-ICP METHOD
PB I	LEAD-ICP METHOD
SE F	SELENIUM-FURNACE METHOD
T P	TOTAL PHOSPHORUS

POST, BUCKLEY, SCHUH & JERNIGAN, INC.
[illegible] COLONIAL DR
ORLANDO, FLORIDA [illegible]

ge 2 PBS&J ENV LABS REPORT Work Order # 90-05-163
ceived: 05/18/90 Results By Test

TEST CODE default units	Sample 01 (entered units)	Sample 02 (entered units)	Sample 03 (entered units)
AG_F ug/l	<1 MG/KG	<1 MG/KG	1.1400 MG/KG
AS_F mg/l	<5 MG/KG	<5 MG/KG	<5 MG/KG
BA_I mg/l	<15.0 MG/KG	15.0000 MG/KG	48.5000 MG/KG
CD_I mg/l	2.270 MG/KG	1.330 MG/KG	<1 MG/KG
CR_I mg/l	99.600 MG/KG	113.000 MG/KG	50.900 MG/KG
HG mg/l	<0.02 MG/KG	<0.02 MG/KG	<0.02 MG/KG
NA_I mg/l	502.000 MG/KG	426.000 MG/KG	171.000 MG/KG
PB_I mg/l	<5 MG/KG	<5 MG/KG	<5 MG/KG
SE_F mg/l	<1 MG/KG	<1 MG/KG	<1 MG/KG
T_P mg/l as P	13700.00 MG/KG WET WGHT.	15900.00 MG/KG WET WGHT.	17400.00 MG/KG WET WGHT.

POST, BUCKLEY, SCHUH & JERNIGAN, INC.
4611 E COLONIAL DR
ORLANDO, FLORIDA 32807

ceived: 05/18/90

PBS&J ENV LABS REPORT Work Order # 90-05-163
Results By Test

SAMPLE Sample Id	Test: AG F ug/l	Test: AS F mg/l	Test: BA I mg/l	Test: CD I mg/l	Test: CR I mg/l
01 CLAY SAMPLE 1	<1 MG/KG	<5 MG/KG	<15.0 MG/KG	2.270 MG/KG	99.600 MG/KG
02 CLAY SAMPLE 2	<1 MG/KG	<5 MG/KG	15.0000 MG/KG	1.330 MG/KG	113.000 MG/KG
03 CLAY SAMPLE 3	1.1400 MG/KG	<5 MG/KG	48.5000 MG/KG	<1 MG/KG	50.900 MG/KG

SAMPLE Sample Id	Test: HG mg/l	Test: NA I mg/l	Test: PB I mg/l	Test: SE F mg/l	Test: T P mg/l as P
01 CLAY SAMPLE 1	<0.02 MG/KG	502.000 MG/KG	<5 MG/KG	<1 MG/KG	13700.00 MG/KG WET WGHT.
02 CLAY SAMPLE 2	<0.02 MG/KG	426.000 MG/KG	<5 MG/KG	<1 MG/KG	15900.00 MG/KG WET WGHT.
03 CLAY SAMPLE 3	<0.02 MG/KG	171.000 MG/KG	<5 MG/KG	<1 MG/KG	17400.00 MG/KG WET WGHT.

AMPLE ID CLAY SAMPLE 1 SAMPLE # 01 FRACTIONS: A,B
Date & Time Collected 05/18/90 08:30:00 Category

G_F	<1	AS_F	<5	BA_I	<15.0	CD_I	2.270	CR_I	99.600	HG	<0.02
	MG/KG		MG/KG		MG/KG		MG/KG		MG/KG		MG/KG
IA_I	502.000	PB_I	<5	SE_F	<1	T_P	13700.00				
	MG/KG		MG/KG		MG/KG		MG/KG WET WGHT.				

AMPLE ID CLAY SAMPLE 2 SAMPLE # 02 FRACTIONS: A,B
Date & Time Collected 05/18/90 08:35:00 Category

G_F	<1	AS_F	<5	BA_I	15.0000	CD_I	1.330	CR_I	113.000	HG	<0.02
	MG/KG		MG/KG		MG/KG		MG/KG		MG/KG		MG/KG
IA_I	426.000	PB_I	<5	SE_F	<1	T_P	15900.00				
	MG/KG		MG/KG		MG/KG		MG/KG WET WGHT.				

AMPLE ID CLAY SAMPLE 3 SAMPLE # 03 FRACTIONS: A,B
Date & Time Collected 05/18/90 08:40:00 Category

G_F	1.1400	AS_F	<5	BA_I	48.5000	CD_I	<1	CR_I	50.900	HG	<0.02
	MG/KG		MG/KG		MG/KG		MG/KG		MG/KG		MG/KG
IA_I	171.000	PB_I	<5	SE_F	<1	T_P	17400.00				
	MG/KG		MG/KG		MG/KG		MG/KG WET WGHT.				

POST, BUCKLEY, SCHUH & JERNIGAN, INC.
6635 E. COLONIAL DR.
ORLANDO, FLORIDA 32807

e 1 PBS&J ENV LABS REPORT Work Order # 90-06-001
eived: 06/01/90 06/12/90 15:17:48

EPORT TO: LAKE COUNTY LANDFILL-ASTATULA
PBS&J ORLANDO
800 N. MAGNOLIA AVE SUITE 600
ORLANDO, FL. 32803
ATTEN OMAR SMITH

LIENT: LAKE COUNTY SAMPLES 3
MPANY: LAKE COUNTY LANDFILL-ASTATULA
ILITY:

RK ID: CLAY SAMPLES
TAKEN: PROJECT NO. 07-568.00
TRANS:
TYPE:
.O. #:
VOICE: under separate cover

PREPARED BY: PBS&J Environmental Lab.
6635 East Colonial Drive
Orlando, Florida 32807
DHRS/DER# 83170, E83011
ATTEN: Kimberly Kunihiro
PHONE: (407) 277-4443 AIHA # 213

K C Kunihiro
CERTIFIED BY

CONTACT KUNIHIRO

We are pleased to provide this report of analysis. If you have any questions regarding this report or further analysis please feel free to telephone.

SAMPLE IDENTIFICATION

RED CLAY SAMPLE 1
RED CLAY SAMPLE 2
NATURAL SANDS

TEST CODES and NAMES used on this report

AG F	SILVER-FURNACE METHOD
AS F	ARSENIC-FURNACE METHOD
BA I	BARIUM-ICP METHOD
CD I	CADMIUM-ICP METHOD
CR I	CHROMIUM-ICP METHOD
HG	MERCURY
NA I	SODIUM-ICP METHOD
PB I	LEAD-ICP METHOD
SE F	SELENIUM-FURNACE METHOD
T P	TOTAL PHOSPHORUS

POST, BUCKLEY, SCHUH & JERNIGAN, INC.

e 2 PBS&J ENV LABS REPORT Work Order # 90-06-001
eived: 06/01/90 Results By Test

EST CODE efault units	Sample 01 (entered units)	Sample 02 (entered units)	Sample 03 (entered units)
G_F	<1	<1	1.1300
g/l	mg/kg	mg/kg	mg/kg
S_F	<5	7.41	<5
g/l	mg/kg	mg/kg	mg/kg
A_I	<15.0	<15.0	<15
g/l	mg/kg	mg/kg	mg/kg
D_I	<1.0	1.050	<1.0
g/l	mg/kg	mg/kg	mg/kg
R_I	18.900	28.800	<4
g/l	mg/kg	mg/kg	mg/kg
G	<0.02	0.0260	<0.02
g/l	mg/kg	mg/kg	mg/kg
A_I	13.000	11.800	10.100
g/l	mg/kg	mg/kg	mg/kg
B_I	<5.0	<5.0	<5
g/l	mg/kg	mg/kg	mg/kg
E_F	<1	<1	<1
g/l	mg/kg	mg/kg	mg/kg
_P	280.00	274.00	84.80
g/l as P	MG/KG WET WT.	MG/KG WET WT.	MG/KG WET WT.

POST, BUCKLEY, SCHUH & JERNIGAN, INC.
8615 E. COLONIAL DR.
ORLANDO, FLORIDA 32807

PBS&J ENV LABS REPORT Work Order # 90-06-001
:eived: 06/01/90
Results By Test

SAMPLE Sample Id	Test: AG F ug/l	Test: AS F mg/l	Test: BA I mg/l	Test: CD I mg/l	Test: CR I mg/l
01	<1	<5	<15.0	<1.0	18.900
RED CLAY SAMPLE	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg
02	<1	7.41	<15.0	1.050	28.800
RED CLAY SAMPLE	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg
03	1.1300	<5	<15	<1.0	<4
NATURAL SANDS	mg/kg	mg/kg	mg/kg	mg/kg	mg/kg

SAMPLE Sample Id	Test: HG mg/l	Test: NA I mg/l	Test: PB I mg/l	Test: SE F mg/l	Test: T P mg/l as P
01	<0.02	13.000	<5.0	<1	280.00
RED CLAY SAMPLE	mg/kg	mg/kg	mg/kg	mg/kg	MG/KG WET WT.
02	0.0260	11.800	<5.0	<1	274.00
RED CLAY SAMPLE	mg/kg	mg/kg	mg/kg	mg/kg	MG/KG WET WT.
03	<0.02	10.100	<5	<1	84.80
NATURAL SANDS	mg/kg	mg/kg	mg/kg	mg/kg	MG/KG WET WT.

AMPLE ID RED CLAY SAMPLE 1 SAMPLE # 01 FRACTIONS: A,B
Date & Time Collected 06/01/90 08:30:00 Category

G_F	<1	mg/kg	AS_F	<5	mg/kg	BA_I	<15.0	mg/kg	CD_I	<1.0	mg/kg
CR_I	18.900	mg/kg	HG	<0.02	mg/kg						
A_I	13.000	mg/kg	PB_I	<5.0	mg/kg	SE_F	<1	mg/kg	T_P	280.00	MG/KG WET WT.

AMPLE ID RED CLAY SAMPLE 2 SAMPLE # 02 FRACTIONS: A,B
Date & Time Collected 06/01/90 08:30:00 Category

G_F	<1	mg/kg	AS_F	7.41	mg/kg	BA_I	<15.0	mg/kg	CD_I	1.050	mg/kg
CR_I	28.800	mg/kg	HG	0.0260	mg/kg						
A_I	11.800	mg/kg	PB_I	<5.0	mg/kg	SE_F	<1	mg/kg	T_P	274.00	MG/KG WET WT.

AMPLE ID NATURAL SANDS SAMPLE # 03 FRACTIONS: A,B
Date & Time Collected 06/01/90 08:25:00 Category

G_F	1.1300	mg/kg	AS_F	<5	mg/kg	BA_I	<15	mg/kg	CD_I	<1.0	mg/kg
CR_I	<4	mg/kg	HG	<0.02	mg/kg						
A_I	10.100	mg/kg	PB_I	<5	mg/kg	SE_F	<1	mg/kg	T_P	84.80	MG/KG WET WT.

Omar Smith PBS&J
800 N. Magnolia Blvd
Suite 600
Orlando, Fl 32801
Attn: Omar Smith

Date Reported: 06/07/90
OLI Contact: D_HOLDING

Work ID: 07-568.00 Lk County Landfill
Total Samples: 3
Matrix:

Sampling information is based on data supplied by Client

SAMPLE IDENTIFICATION	TEST CODES and NAMES used on this report
01 #1 Soil	AG FUR Silver
02 #2 Soil	AS FUR Arsenic
03 #3 Soil	BA ICP Barium
	CD ICP Cadmium
	CR ICP Chromium
	HG CV Mercury
	NA ICP Sodium
	PB FUR Lead
	SE FUR Selenium

OLI Florida Department of Health & Rehabilitative Service Identification Numbers are:
Drinking Water Certification Number 83141, Environmental Certification Number E83033

Respectfully Submitted,
ORLANDO LABORATORIES, INC.

Acting
LABORATORY DIRECTOR

Eric Malarek
QUALITY CONTROL

.LANDO LABORATORIES,INC. REPORT OF ANALYSIS Work Order # 90-05-418
eceived: 05/31/90 Results by Sample

SAMPLE ID #1 Soil SAMPLE # 01A
DATE COLLECTED 05/18/90 08:30:00 MATRIX SOIL

AG_FUR	<0.010 mg/l	AS_FUR	<0.010 mg/l	BA_ICP	0.240 mg/l	CD_ICP	0.005 mg/l	CR_ICP	0.015 mg/l	HG_CV	<0.0005 mg/l
NA_ICP	25.0 mg/l	PB_FUR	<0.010 mg/l	SE_FUR	<0.005 mg/l						

SAMPLE ID #2 Soil SAMPLE # 02A
DATE COLLECTED 05/18/90 08:35:00 MATRIX SOIL

AG_FUR	<0.010 mg/l	AS_FUR	<0.010 mg/l	BA_ICP	0.270 mg/l	CD_ICP	0.005 mg/l	CR_ICP	0.017 mg/l	HG_CV	<0.0005 mg/l
NA_ICP	22.0 mg/l	PB_FUR	<0.010 mg/l	SE_FUR	<0.005 mg/l						

SAMPLE ID #3 Soil SAMPLE # 03A
DATE COLLECTED 05/18/90 08:40:00 MATRIX SOIL

AG_FUR	<0.010 mg/l	AS_FUR	<0.010 mg/l	BA_ICP	0.220 mg/l	CD_ICP	<0.005 mg/l	CR_ICP	0.013 mg/l	HG_CV	0.0119 mg/l
NA_ICP	15.0 mg/l	PB_FUR	<0.010 mg/l	SE_FUR	<0.005 mg/l						

Table 1
SUITABILITY OF SLUDGE FOR LAND APPLICATION AND CLASS AA REQUIREMENTS

Heavy Metal	Concentration (dry mg/kg) Suitable	Not Suitable	Class AA Concentration (dry mg/kg)
Cadmium	≤ 100	> 100	≤ 30
Copper	≤ 3,000	> 3,000	≤ 900
Lead	≤ 1,500	> 1,500	≤ 1,000
Nickel	≤ 500	> 500	≤ 100
Zinc	≤ 10,000	> 10,000	≤ 1,800

Table 2
SUMMARY OF LAND APPLICATION CLASSIFICATIONS

Parameter	Class A	Class B	Class C
Requirements			
Stabilization	PFRP [1]	PSRP [2]	PSRP [2] (without meeting all requirements)
Agricultural Use Plan	• Required if sludge application meets crop needs • Update annually	Same	Same
Dedicated Use Plan	• Required if sludge application exceeds crop needs • Update annually	Same	Same
Testing			
Nutrients and Metals	3 months	Same	Same
Soil pH	Annual	Annual	Annual
Recordkeeping	Low	High	High
Land Application			
Suitable Sites	Public access • Playgrounds • Parks • Lawns • Golf Courses	Limited access • Sod farms • Highway shoulders and medians • Plant nurseries • Land reclamation	Same as Class B
Restrictions			
• General public access	None	12 months	12 months
• Harvest pasture vegetation	None	30 days	30 days
• Livestock grazing	None	30 days	30 days
• Harvest human food not in contact with soil	None	30 days	60 days
• Use on root crops, leafy vegetables, etc.	None	Prohibited	Prohibited

[1] Process to Further Reduce Pathogens (PFRP) as defined by 40 CFR 257
[2] Process to Significantly Reduce Pathogens (PSRP) as defined by 40 CFR 257

Table 3

SUMMARY OF PSRP AND PFRP SLUDGE TREATMENT PROCESS

Processes to Significantly Reduce Pathogens (PSRP)

Treatment Processes	Requirements
Aerobic Digestion	60 days @ 15° C or 40 days @ 20° C
Air Drying	3 months
Anaerobic Digestion	60 days @ 20° C or 15 days @ 35°-55° C
Composting	5 days @ 40° C and 4 hours @ 55° C
Lime Stabilization	pH of 12 for 2 hours

Processes to Further Reduce Pathogens (PFRP)

Treatment Processes	Requirements
Composting	3 days @ 55° C (15 days for Windrow)
Heat Drying	80° C; 10% moisture
Heat Treatment	30 minutes @ 180° C
Irradiation	Beta or Gamma
Pasturization	30 minutes @ 70° C
Thermophilic Aerobic Digestion	10 days @ 55°-60° C

Note: As defined in 40 CFR,Part 257

Table 4

AGRONOMIC APPLICATION RATES AND ESTIMATED REQUIREMENTS

Crop(1)	Nitrogen [1] Application Rate (lb/acre/year)	Estimated [2] Maximum Wastewater Residual Application Rate (lb/acre/day)	Estimated [3] Land Requirement (acre)
Field Crops			
Citrus	100	9.1	149
Corn	200	18.3	74
Cotton	120	11.0	124
Grain Sorgham	100	9.1	149
Oats	60	5.5	247
Peanuts	—	—	—
Soybeans	—	—	—
Sugarcane	200	18.3	74
Wheat	125	11.4	119
Forage Crops			
Alfalfa Hay	—	—	—
Bahiagrass - Hay or Grazed	160	14.6	93
Bermudagrass - Hay or Grazed	250	22.8	59
Clover-grass-Hay	100	9.1	149
Clover-grass-Grazed	—	—	—
Guineagrass	300	27.4	49
Johnsongrass	700	63.7	21
Limpograss	400	36.5	37
Napiergrass	400	36.5	37
Pangolagrass	350	32.0	42
Paragrass	350	32.0	42
Bygrass-Grazed	200	18.3	74
Sorgham & Sudan hybrid	400	36.5	37

1. As shown in FAC 17-640
2. Based on 6% total nitrogen; 50% available, 7-day week
3. Based on Influent BOD of 250 mg/l and solids generation rate of 0.65 lb/lb BOD

Table 5
APPLICATION RATES OF RESIDUALS BASED ON
HEAVY METAL LOADING LIMITS

Parameter	Typical Concentration (mg/kg)	Cumulative [2] Loading Limit (lb/acre)	Residuals [3] Application Rate (lb/acre/day)
Cadmium [1]	15	4.4	91
Copper	500	125	34
Lead	80	500	856
Nickel	50	125	342
Zinc	800	250	43

Notes:
1. Application rate based on maximum yearly loading of 0.5 lb/ac/yr
2 As stated in FAC 17-640
3 Based on 20-year site life and 7-day week

APPENDIX C
PROJECT NO. 1: SOUTH WEST ALACHUA LANDFILL TOP COVER
(CLASSES 1 AND 3)
(WITH PERMISSION OF MFM AND ALACHUA COUNTY AUTHORITY)

NATURAL CLAY LINING AND COVER

1. SCOPE

1.1 WORK INCLUDED: Work necessary to furnish and install the natural clay lining and cover. The NATURAL CLAY LINING AND COVER is presented as an alternate to using SOIL-BENTONITE LINING AND COVER. The Contractor may select either material for this construction.

1.2 DEFINITIONS:

1.2.1 Natural Clay: "Natural clay" is a fine-grained material having a low hydraulic conductivity that is imported and compacted to be used as a soil lining or cover. The material may be classified as CH, CL, SC, or MH, using the Unified Soil Classification System, ASTM D 2487.

1.2.2 Imported Clay: Natural clay material which meets the requirements of this Specification and is obtained offsite and transported to the project site.

1.2.3 Relative Compaction: The ratio, in percent, of the as-compacted field dry density to the laboratory maximum dry density as determined by ASTM D 1557.

1.2.4 Optimum Moisture Content: Determined by the ASTM standard specified to determine the maximum dry density for relative compaction.

1.3 SUBMITTALS:

1.3.1 Submittals shall be made to the Engineer in accordance with the General Conditions, Section GENERAL REQUIREMENTS, and the requirements of this section.

1.3.2 Contractor shall provide the following submittals to the Engineer at least 20 calendar days before the material is required for use:

1.3.2.1 Identification and location of proposed clay borrow sources.

1.3.2.2 Laboratory test results from at least three clay samples taken from the proposed primary borrow source(s) including Atterberg limits, gradation (sieve and hydrometer analysis), natural moisture content, laboratory permeability, and moisture-density relationship. All tests supporting this submittal shall be performed by the soil testing company accepted by the Engineer.

1.3.2.3 Proposed clay preparation and installation equipment, including equipment manufacturer's name, equipment operating statistics, and dimensions.

1.3.2.4 Construction Quality Assurance Plan for clay lining and cover.

1.3.2.5 Proposed method of protecting completed work or controlling moisture to prevent drying, cracking, saturation, or other damage prior to installation of a permanent covering.

1.3.2.6 Stormwater control plan.

1.3.3 Quality Control Submittals:

1.3.3.1 Furnish the following:

1.3.3.1.1 Qualifications of independent soil testing company including equipment and experience for back pressure saturated permeability testing, description of proposed laboratory testing equipment, and procedures.

1.3.3.1.2 Certification, test results, source, and samples for all natural clay material, including results of gradation tests, Atterberg limits tests, Moisture-Density Relationship and permeability tests.

1.3.3.2 Imported Natural Clay Material: Certification that material meets specified requirements and test results from qualified independent soil testing company.

1.3.3.3 Catalog and manufacturer's data sheets for compaction equipment.

1.3.3.4 Results of quality control tests including field density, moisture content, and permeability tests for completed clay lining or cover.

1.3.3.5 Gradation and Moisture Content Test Results for Imported Material within 48 hours after sampling.

1.3.3.6 Topographic Surveys: Furnish within 48 hours after completion of subgrade preparation and again after natural clay layer placement.

1.4 QUALITY ASSURANCE:

1.4.1 Codes, Ordinances, and Statutes: Comply with applicable codes, ordinances, and statutes, including those pertaining to borrow pit operations or hauling of materials on public roads, and bear sole responsibility for any penalties imposed for noncompliance.

1.4.2 Quality Assurance refers to all activities designed to provide adequate confidence that materials and workmanship meet the requirements necessary to fulfill the project objectives. In a larger sense, Quality Assurance includes Quality Control performed by the contractor or supplier as well as Quality Assurance tests and procedures performed by Quality Assurance personnel.

1.5 QUALITY CONTROL: Quality Control refers to those actions taken by the manufacturer, fabricator, or contractor to ensure that materials and workmanship met the requirements of the contract or purchase order, and the applicable drawings and specifications.

2. MATERIALS

2.1 NATURAL CLAY:

2.1.1 Natural clay materials specified in this section are subject to the following requirements:

2.1.1.1 Conduct tests, including grain size, Atterberg limits, moisture content, density, and permeability tests, as necessary to locate and confirm an acceptable source of imported material.

2.1.1.2 Conduct minimum three sets of above tests on samples taken across source area, to proposed source depth, taking into account variability of soils within the source.

2.1.1.3 Do not deliver imported materials to the site until the proposed source and materials tests have been tentatively accepted in writing by Engineer.

2.1.1.4 Final acceptance will be based on tests made on samples of material taken from the completed and compacted layer.

2.1.1.5 Perform all testing using a qualified independent soil testing company accepted by the Engineer.

2.1.1.6 Testing to include gradation (ASTM D 1140), Plasticity (ASTM D 4318), and Moisture Content (ASTM D 2216) Tests:

2.1.1.6.1 Perform using qualified soil testing company on samples of the natural clay material prior to delivery of material to site at minimum frequency of one test for every 1,500 tons of material.

2.1.1.6.2 Conduct tests more often as determined by Engineer if variation in test results is occurring, or if material appears to depart from Specifications.

2.1.1.7 If tests indicate material does not meet Specification requirements, terminate material placement until corrective measures are taken.

2.1.1.8 Remove and replace material which does not meet Specification requirements that is placed in the work at no additional compensation.

2.1.2 Composition: Material having 80 percent by weight passing the U.S. No. 4 sieve and at least 30 percent passing a U.S. No. 200 sieve.

2.1.3 Properties:

2.1.3.1 Liquid Limit: 30 to 50 maximum.

2.1.3.2 Plasticity Index: 20 to 50.

2.1.3.3 Permeability: Less than 1 times 10^{-7} centimeters per second when tested by an independent soil testing company accepted by the Engineer.

2.2 WATER FOR COMPACTION: Furnish as required.

2.3 COMPACTION EQUIPMENT:

2.3.1 The compaction equipment shall be suitable type and adequate to obtain densities specified, and shall provide satisfactory breakdown of materials to form a dense fill.

2.3.2 The compaction equipment shall be maintained and operated in a condition that will deliver manufacturer's rated compactive effort. If inadequate densities are obtained, larger and/or different types of additional equipment will be provided.

2.3.3 Hand-Operated Equipment: Shall be capable of achieving specified densities.

2.3.4 Equipment Types:

2.3.4.1 Natural clay materials shall be compacted using equipment that provides a kneading action, such as a wobble-wheeled roller or a sheepsfoot roller having tines at least 4 inches long such as Caterpillar 815.

2.3.4.2 If a sheepsfoot is used, the finished surface of the final lift shall be rolled with a smooth steel drum roller or rubber-tired roller to eliminate tine or roller marks and provide a smooth, dense surface.

2.4 MOISTURE CONTROL EQUIPMENT: Equipment for applying water shall be of a type and quality adequate for the work, shall not leak, and shall be equipped with a distributor bar or other approved device to assure uniform application. Equipment for mixing and drying out material shall consist of blades, discs, or other equipment approved

by the Engineer. Mixing of natural clay may also be required to get even distribution of moisture.

2.5 LABORATORY PERMEABILITY TESTING:

2.5.1 Laboratory permeability tests shall be made by the soil testing company accepted by the Engineer on samples of natural clay material.

2.5.2 Samples taken from the proposed source shall be prepared by compacting to 90 percent plus or minus 2 percent relative compaction at a moisture content within the range of optimum to optimum plus 3 percent.

2.5.3 Samples taken from the compacted and completed clay lining or cover, or from completed layers thereof, shall be obtained using a thin-walled Shelby tube sampler and prepared by extruding and trimming the sample. All holes shall be plugged using a bentonite seal.

2.5.4 Constant head triaxial permeability tests shall be conducted in accordance with Corps of Engineers Method EM-1110-1906, EPA Method 9100 guidelines to measure the permeability of the clay material. Test specimens shall be trimmed from the sample to a length-to-diameter ratio of 0.5 to 1.3. Specimens shall be sheathed in a latex membrane, placed in a triaxial cell, consolidated under an average effective confining pressure of 3 to 5 psi, subjected to a back pressure sufficient to saturate the specimen, and permeated under a hydraulic gradient less than 30 across the specimen. Time and flow data shall be recorded for at least one day beyond the time when inflow rate equals the outflow rate, at which time the pressures may be relieved and physical measurements of the specimens obtained for calculations. The soil testing company shall submit proposed equipment description and testing procedures to the Engineer for review.

2.5.5 As a minimum, the following data shall be submitted to the Engineer with the results of each permeability test:

2.5.5.1 Dates testing commenced and stopped.

2.5.5.2 Sample number and location.

2.5.5.3 Sampling method.

2.5.5.4 Specimen length and diameter.

2.5.5.5 Specimen dry unit weight and moisture content.

2.5.5.6 Hydraulic gradient.

2.5.5.7 Maximum cell pressure and back pressure.

2.5.5.8 A graph plotting flow volume versus time for inflow and outflow.

2.5.5.9 Degree of saturation.

2.5.5.10 Measured coefficient of permeability.

2.5.5.11 Comments.

2.5.5.12 Initials (with date) of lab personnel performing the test.

2.5.5.13 Name and signature (with date) of person performing quality assurance check for the soil testing company.

2.5.5.14 Name and signature (with date) of Contractor's quality control manager for this project.

2.6 CODES, ORDINANCES, AND STATUTES: The Contractor shall become familiar with, and comply with, all applicable codes, ordinances, and statutes, including those pertaining to borrow pit operations or hauling of materials on public roads, and shall bear sole responsibility for any penalties imposed for noncompliance.

3. WORKMANSHIP

3.1 CONTROL OF WATER:

3.1.1 Keep working surfaces, excavations and trenches free of water during placing of fill material and at such other times as required for efficient and safe execution of the work.

3.1.2 Develop and implement a stormwater control plan to control surface water runoff and run-on. Temporary berms, ditches, or diversions may be used.

3.1.3 Implement erosion control practices to protect the exposed surface from gulleying, surface wash, or other erosion. Provide measures such as straw bales, silt fences, temporary slope flumes, or other methods to protect the work. Repair damaged or washed-out areas including replacement of natural clay or other materials at the Contractor's expense.

3.2 SITE GRADING: Perform all earthwork to the lines and grades established by the Engineer, with proper allowance for topsoil or overlying materials where specified. Shape, trim, and finish slopes to conform with the lines, grades, and cross sections given. Make slopes free of all exposed roots and stones exceeding 1-inch diameter that are loose and liable to fall. Round tops of banks to circular curves, in general, not less than a 6-foot radius. Rounded surfaces shall be neatly and smoothly trimmed. Overexcavating and backfilling to the proper grade will not be acceptable. Finished site grading will be reviewed by the Engineer.

3.3 CLAY LINING OR COVER TEST FILL:

3.3.1 The Contractor shall construct a test fill to investigate optimal construction equipment and procedures for the clay lining or cover. Results of the test fill will be reviewed by the Engineer to determine which construction procedures produce a dense, homogeneous, layer of clay with a hydraulic conductivity at or below the specified level and that is free of cracks and visible clods. The Engineer will then notify the Contractor of the detailed equipment and procedures to be used in constructing the clay lining or cover.

3.3.2 The test fill shall be designed and constructed to investigate the following:

3.3.2.1 The soil screening and pulverizing procedures needed to properly process the clay prior to compaction.

3.3.2.2 The moisture content of the clay prior to compaction.

3.3.2.3 The lift thicknesses, compaction procedures, and number of passes for the proposed compaction equipment.

3.3.2.4 The dry unit weight achieved and measured by field density testing.

3.3.3 The Contractor shall prepare and submit a plan to the Engineer describing the procedures and schedule for the test fill. The test fill plan shall be submitted to the Engineer for comment at least 20 working days prior to starting the test fill. As a minimum, test fill construction shall include the following:

3.3.3.1 Prepare an area for the test fill to meet the requirements specified herein for subgrade preparation.

3.3.3.2 Provide adequate clay to prepare a 100-foot by 100-foot clay pad with the thickness equal to the clay lining or cover shown on the Drawings.

A. The Contractor shall select the number of passes anticipated to be adequate for the roller or compactor to be used on the project to achieve the specified compaction for each lift. Compacted lift thicknesses shall be no greater than 8 inches.

B. Upon completion of the test fill, the soil testing company shall perform tests for field density, moisture content, and laboratory or field permeability. In addition, the soil testing company shall conduct one laboratory test for moisture-density relationship for the clay material used in the test fill.

C. The Contractor shall submit these results to the Engineer for review.

3.4 NATURAL LINER CONSTRUCTION:

3.4.1 Placement:

3.4.1.1 Place natural clay after preparation of subgrade in maximum 8-inch lifts.

3.4.1.2 Remove sand or silt inclusions and replace with natural clay material.

3.4.1.3 Compact clay to minimum density of 95 percent relative compaction at a moisture content range of optimum moisture to 5 percentage points above optimum.

3.4.1.4 Overlap joints between adjacent clay panels at least 5 feet.

3.4.1.5 Scarify surface of each lift to depth of 2 inches prior to placing subsequent lift of natural clay material.

3.4.1.6 Surface of Final Lift: Smooth, free from roller marks, holes, depressions more than 1-inch deep, or protrusions extending above the surface more than 1/2 inch.

3.4.1.7 Minimum Thickness of Natural Clay Layer: As shown.

3.4.1.8 Cover or place temporary protective cover of Visqueen over areas on which natural clay layer is exposed within 24 hours of placement of clay.

3.4.1.9 Compact exposed surfaces to protect clay from moisture changes, loss or gain. Wet exposed surfaces as needed to protect clay from cracking.

3.4.1.10 If clay becomes cracked or becomes softened due to moisture changes, scarify full depth of lift and recompact as previously specified.

3.4.2 Do not route construction traffic on top of natural clay lining once material has been placed and compacted.

3.5 MOISTURE CONTROL:

3.5.1 During compacting operations, maintain moisture content in each lift of natural clay material within a range of optimum to plus 5 percentage points above optimum.

3.5.2 If too dry, add water to material by sprinkling the fill, then mixing to make moisture content uniform throughout the lift.

3.5.3 If too wet, aerate material by blading, discing, harrowing, or other methods, to hasten the drying process.

3.6 FIELD QUALITY CONTROL:

3.6.1 Field and Laboratory Testing:

3.6.1.1 Provide services of an independent soil testing company accepted by Engineer to perform quality control testing on completed and compacted clay lining or cover.

3.6.1.2 Field Density and Moisture Content Tests:

3.6.1.2.1 Perform on natural clay layer at a frequency of one test for each 20,000 square feet of each lift.

3.6.1.2.2 Determine in-place density and moisture content by any one or combination of the following methods: ASTM D 2922, D 3017, D 1556, D 2216, or other methods approved by Engineer.

3.6.1.2.3 Cooperate with testing work by leveling and backfilling test areas designated by Engineer.

3.6.1.2.4 If compaction tests indicate density or moisture content is not as specified, terminate material placement and take corrective action prior to continuing placement.

3.6.1.3 Back pressure saturated permeability and Atterberg Limits Tests:

3.6.1.3.1 Include tests for liquid limit, plasticity index, and shrinkage limit.

3.6.1.3.2 Perform on natural clay layer at a frequency of one test for each 40,000 square feet of each lift.

3.6.1.3.3 Perform permeability tests on relatively undisturbed samples obtained with a Shelby tube after compaction of natural clay layer is complete.

3.6.1.3.4 The Contractor shall take undisturbed Shelby tube samples of compacted clay for the permeability tests. The samples shall be tested in a flexible wall permeameter using back pressure saturation by the independent testing company approved by the Engineer.

3.6.1.3.5 If specified permeability of 1 times 10^{-7} centimeters per second for the clay lining and 5 times 10^{-3} centimeters per second for the clay cover is exceeded, excavate defective areas and reconstruct to meet permeability and density requirements.

3.6.1.3.6 Perform such additional work at no additional compensation.

3.6.2 Topographic Surveys: Conduct detailed topographic surveys of the site to document suitable thickness of the installed natural clay layer as follows:

3.6.2.1 Measure elevations on a minimum 50-foot grid over the entire area and at all breaks in grade.

3.6.2.2 Accuracy: Within 0.1 foot vertical and 0.5 foot horizontal in accordance with national surveying standards.

3.6.2.3 As a minimum, conduct surveys at completion of the following tasks:

A. Subgrade preparation

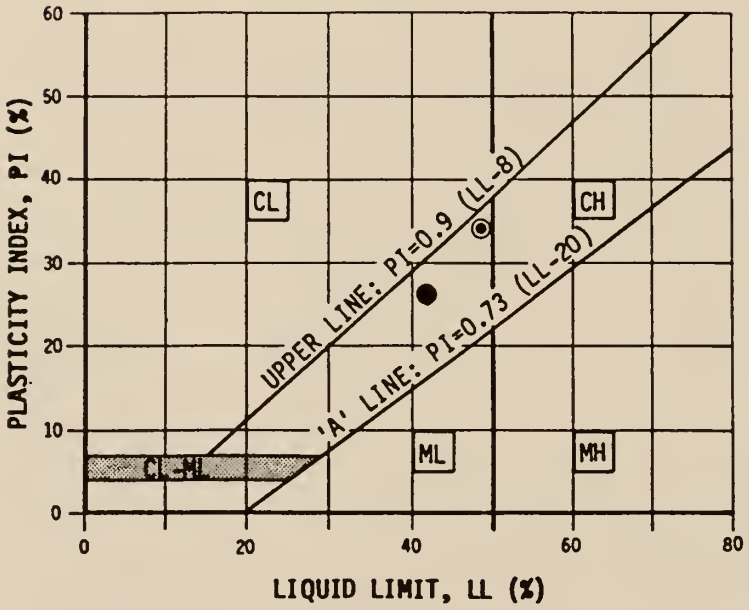
B. Natural clay layer placement

3.6.2.4 Plot results of topographic surveys to a scale of 1 inch equals 50 feet.

4. PAYMENT

4.1 GENERAL: Payment for work in this section will be included as part of the lump sum bid amount stated in the Bid Form.

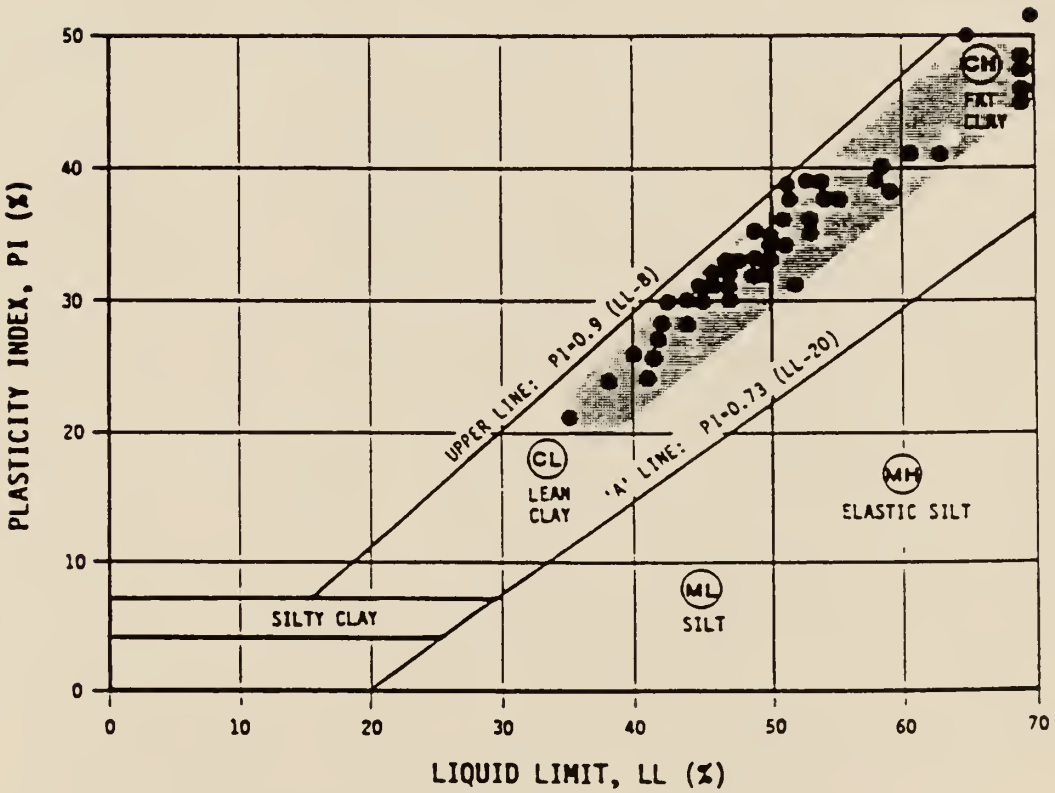
* * * * *

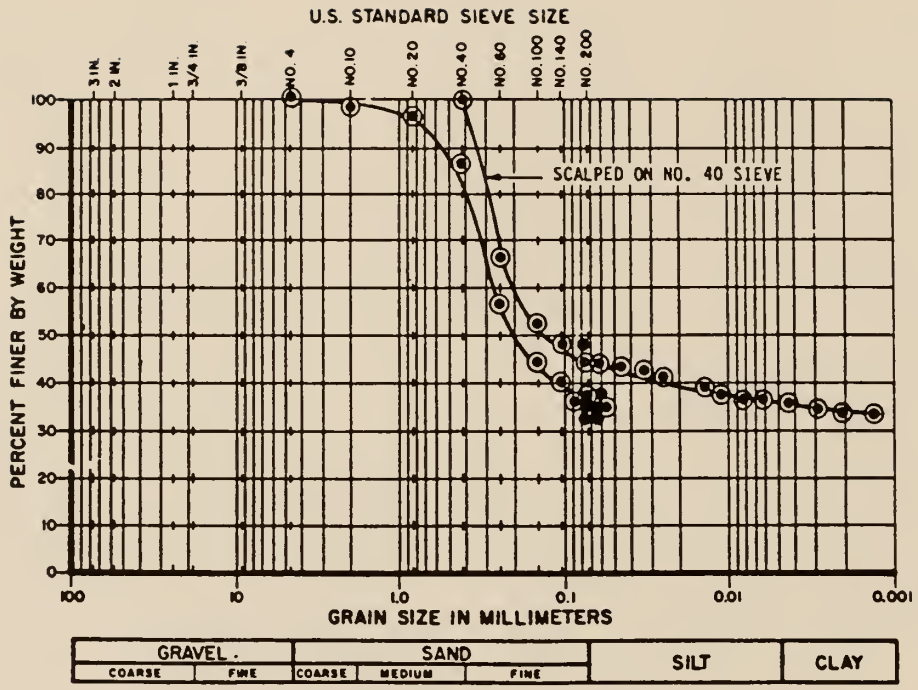


SYMBOL	SAMPLE	LIQUID LIMIT LL(%)	PLASTIC LIMIT PL(%)	PLASTICITY INDEX PI(%)
◉	TEST STRIP	49	15	34
●	MINE SITE	42	16	26

PLASTICITY CHARACTERISTICS OF NATURAL TERRASEAL CALCIUM MONTMORILLONITE-QUARTZ MIXTURE

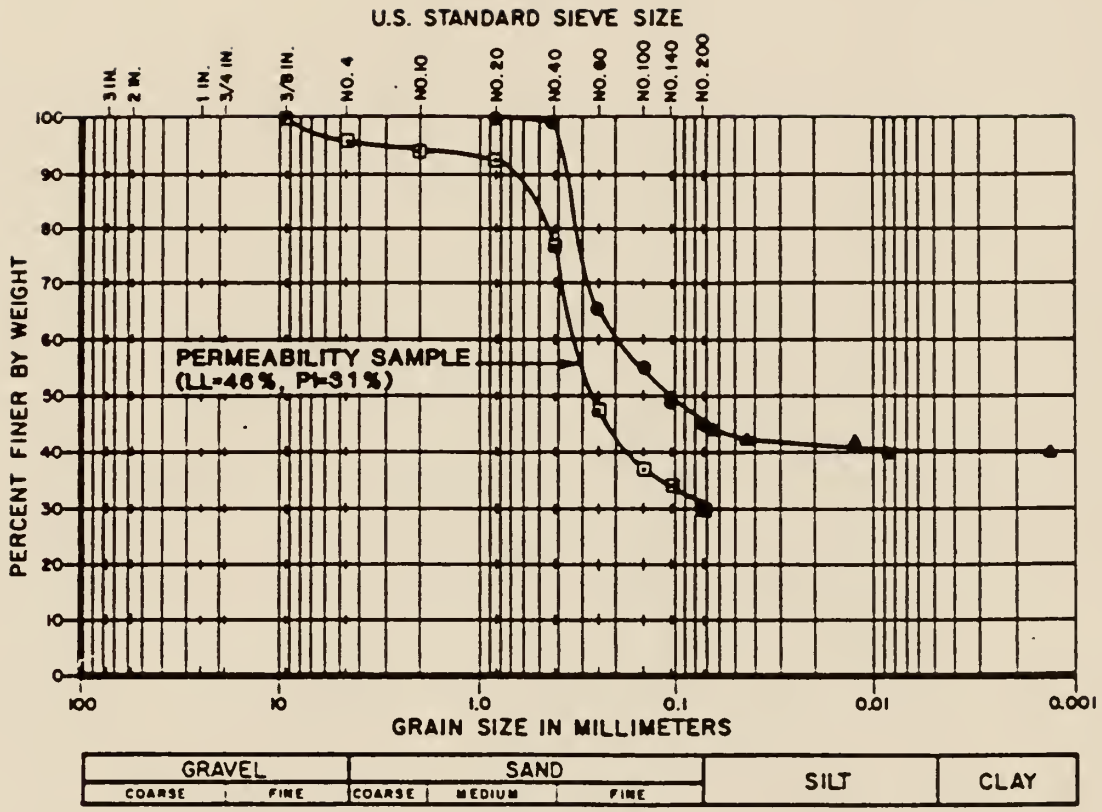
PLASTICITY CHARACTERISTICS OF NATURAL PREMIX TERRASEAL





SYMBOL	SAMPLE	FINES CONTENT, -200(%) HIGH	LOW	AVERAGE
◉	TEST STRIP	38	34	36
●	MINE SITE	47	32	36

PARTICLE SIZE DISTRIBUTION OF NATURAL TERRASEAL CALCIUM MONTMORILLONITE-QUARTZ MIXTURE



PARTICLE SIZE DISTRIBUTION OF NATURAL PRE-MIX TERRASEAL SAMPLE 3

PARTICLE SIZE DISTRIBUTION OF NATURAL PREMIX TERRASEAL

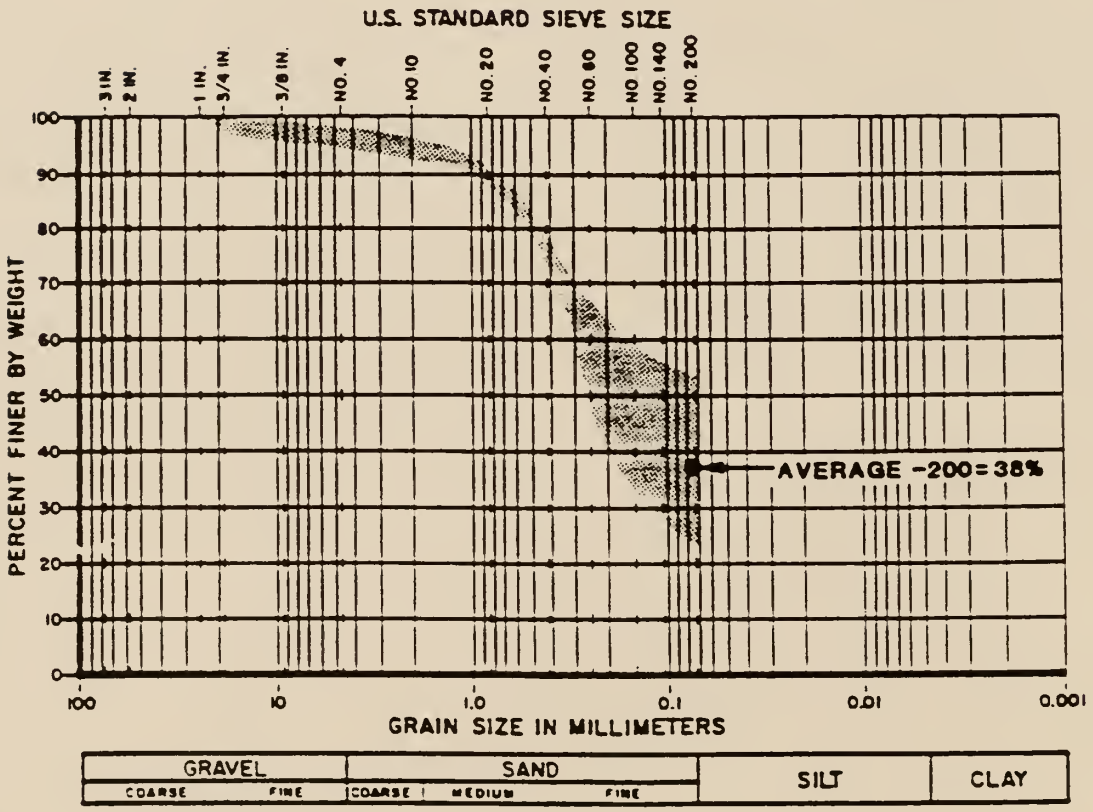


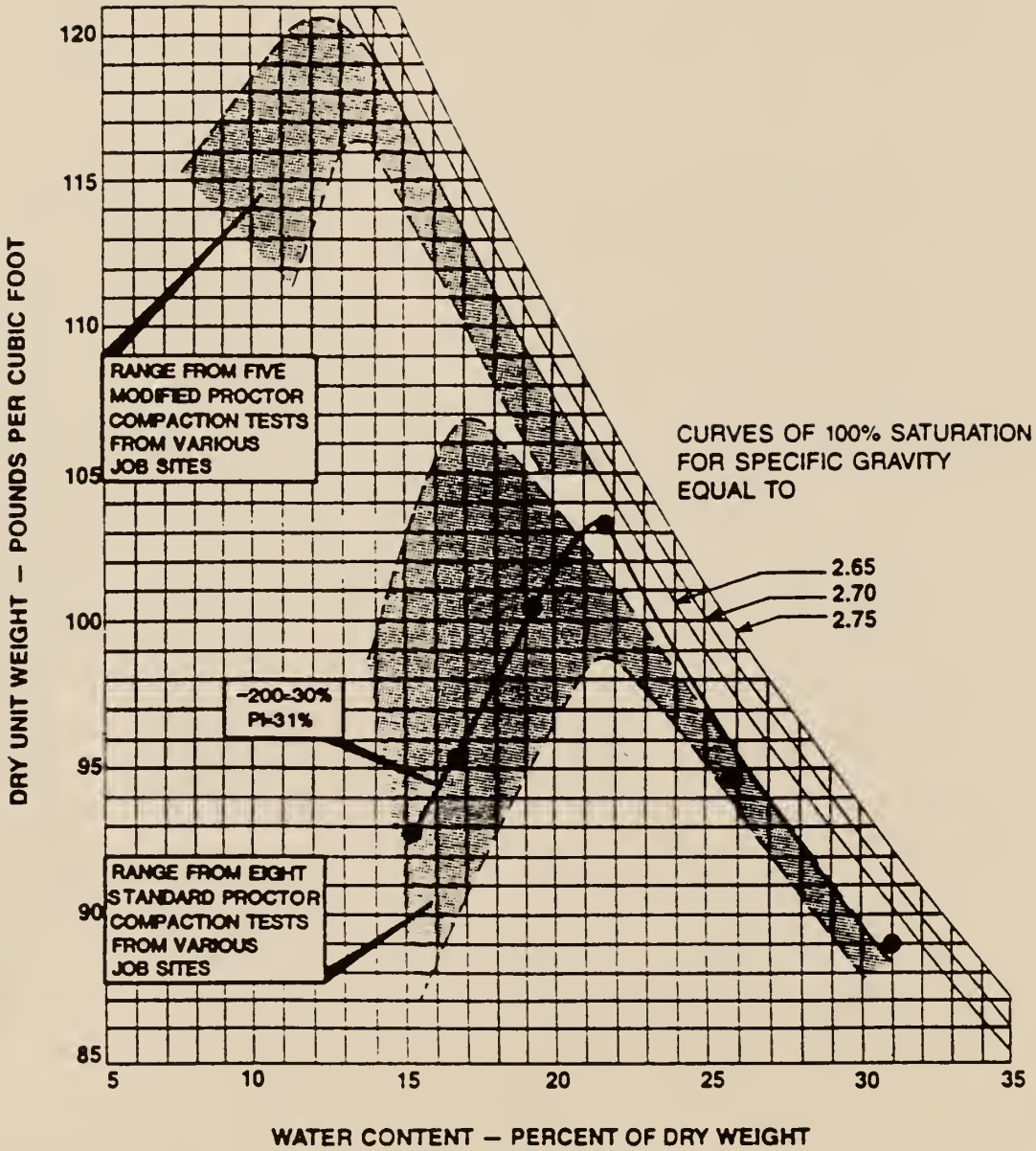
Table 2

SUMMARY OF THICKNESS MEASUREMENTS AND FINES CONTENT DETERMINATIONS ON ALACHUA COUNTY SOUTHWEST LANDFILL COMPACTED CLAY TOP COVER

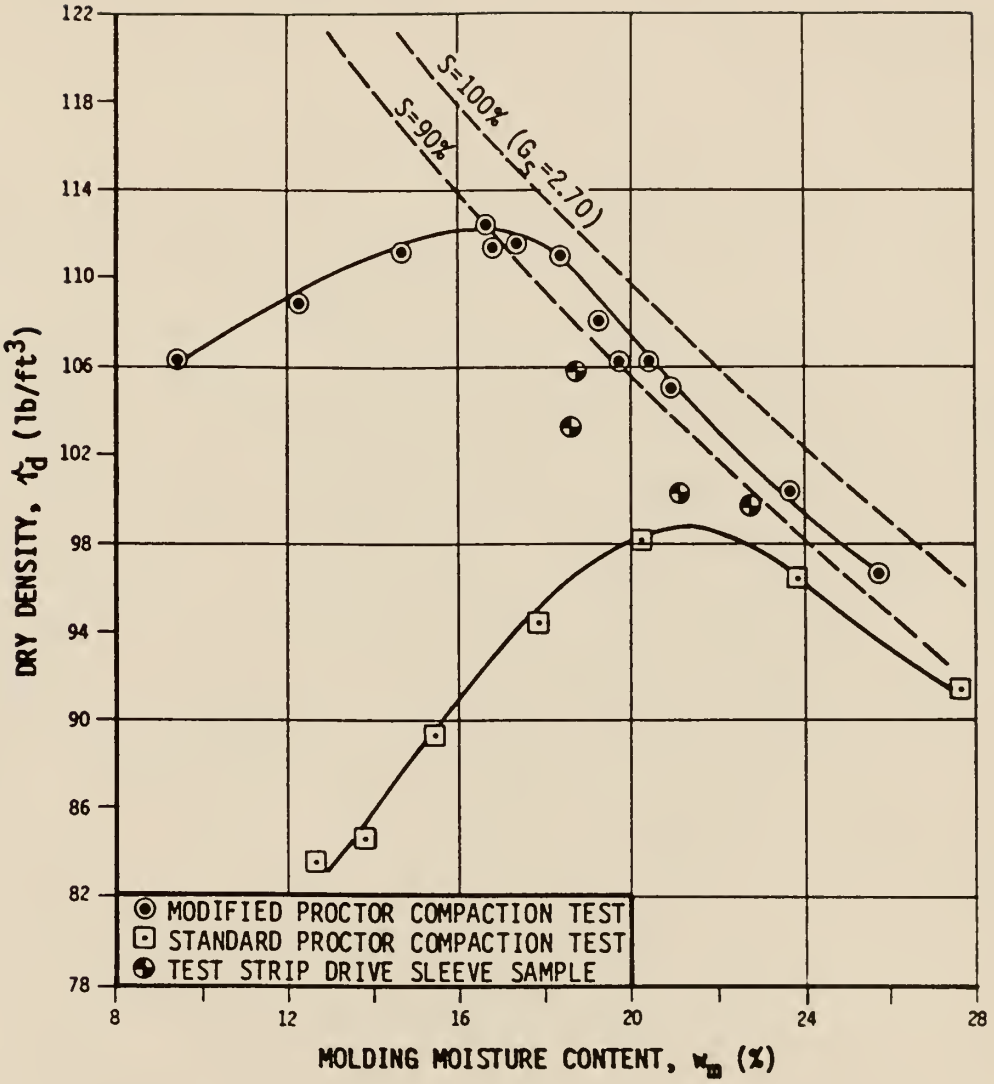
Sample	Coordinates		Sampling Date	Top Cover Thickness (inches)	Fines Content, -200 (%)*	
	North	East			0"-8"	0"-6"
Area II-A	6467	14,532	02-12-87	8.0	41.1	39.5
Area VII-E	6898	15,361	02-14-87	9.0	40.3	-
Area VII-F	7014	15,389	02-14-87	8.5	40.1	42.1
Area VII-G	7150	15,390	02-14-87	9.5	40.6	40.0
Area VIII-A	7150	15,205	02-14-87	8.7	42.6	39.1
Area VIII-B	7165	14,890	02-15-87	9.7	43.1	41.8
Area VII-D	6777	15,366	02-17-87	9.5	39.9	42.7
Area II-B	6449	14,688	02-18-87	9.7	39.6	43.6
Area II-C	6410	14,809	02-18-87	11.0	42.3	41.8
Area IV-A	6672	14,762	02-20-87	12.0	41.8	45.6
Area I-A	6588	14,476	02-24-87	9.0	36.4	42.8
Area IV-C	6725	14,517	02-24-87	8.0	39.1	49.4
Area IV-D	6846	14,522	02-24-87	8.7	35.4	41.4
Area II-D	6435	14,943	02-26-87	8.0	38.9	46.9
Area IV-E	6957	14,539	03-03-87	8.2	35.8	36.6
Area IV-F	7051	14,521	03-03-87	9.0	33.8	37.9
Area IV-G	7167	14,627	03-05-87	9.0	36.6	39.1
Area VIII-C	7250	14,585	03-05-87	9.2	38.1	39.1
Area IV-B	6505	14,903	03-11-87	11.0	40.4	**
Area II-G	6488	15,164	03-12-87	9.7	42.1	39.1
Area II-E	6410	15,150	03-12-87	11.0	41.4	36.0
Area II-F	6396	15,335	03-14-87	8.7	39.8	**
Area VI-A	6402	15,500	03-14-87	10.2	40.2	**
Area VI-B	6513	15,501	03-17-87	10.2	36.1	**
Area VI-C	6681	15,523	03-18-87	10.0	38.9	**
Area VI-D	6816	15,524	03-20-87	8.2	36.9	**

*Fines content from 0"-8" determined on composite soil sample from drive cylinder samples obtained from the sampling depths of 0"-4" and 4"-8". Fines content from 0"-6" determined on drive cylinder sample obtained from sampling depth of 0"-6".

**Fines content from 0"-6" to be reported at a later date.



STANDARD PROCTOR COMPACTION TEST RESULTS ON NATURAL PRE-MIX TERRASEAL SAMPLE 3



MODIFIED AND STANDARD PROCTOR COMPACTION TEST RESULTS AND COMPARISON WITH TEST STRIP IN-SITU DRIVE SLEEVE SAMPLES

Table 1

SUMMARY OF PERMEABILITY TEST RESULTS ON ALACHUA COUNTY SOUTHWEST LANDFILL COMPACTED CLAY TOP COVER

Sample	Coordinates North	Coordinates East	Depth (inches)	Sampling Date	Moisture Content w_m (%)	Dry Density γ_d(lb/ft^3)	Degree of Compaction* γ_d/γ_{dmax}(%)	Fines Content -200 (%)	Coefficient of Permeability k (cm/sec)
Area II-A	6467	14,532	0.0-6.0	02-12-87	19.5	107.4	108.7	39.5	5.5×10^{-9}
Area VII-E	6898	15,361	0.0-6.0	02-14-87	27.9	92.9	94.0	-	1.5×10^{-8}
Area VII-F	7014	15,389	0.0-6.0	02-14-87	24.8	98.0	99.2	42.1	1.1×10^{-8}
Area VII-G	7150	15,390	0.0-6.0	02-14-87	22.8	102.8	104.0	40.0	2.1×10^{-9}
Area VIII-A	7150	15,205	0.0-6.0	02-14-87	22.5	103.0	104.3	39.1	2.4×10^{-8}
Area VIII-B	7165	14,890	0.0-6.0	02-15-87	23.6	101.7	102.9	41.8	1.0×10^{-8}
Area VII-D	6777	15,366	0.0-6.0	02-17-87	22.7	102.9	104.1	42.7	5.7×10^{-9}
Area II-B	6449	14,688	0.0-6.0	02-18-87	23.2	104.3	105.6	43.6	4.7×10^{-9}
Area II-C	6410	14,809	0.0-6.0	02-18-87	20.4	105.5	106.8	41.8	2.1×10^{-8}
Area IV-A	6672	14,762	0.0-6.0	02-20-87	24.4	101.3	102.5	45.6	7.7×10^{-9}
Area I-A	6588	14,476	0.0-6.0	02-24-87	27.6	92.3	93.4	42.8	6.5×10^{-9}
Area IV-C	6725	14,517	0.0-6.0	02-24-87	26.5	93.2	94.3	49.4	8.3×10^{-9}
Area IV-D	6846	14,522	0.0-6.0	02-24-87	25.4	98.9	100.0	41.4	5.3×10^{-9}
Area II-D	6435	14,943	0.0-6.0	02-26-87	23.3	99.9	101.1	46.9	1.0×10^{-8}
Area IV-E	6957	14,539	0.0-6.0	03-03-87	24.0	99.8	101.0	36.6	5.8×10^{-9}
Area IV-F	7051	14,521	0.0-6.0	03-03-87	23.4	98.5	99.7	37.9	5.1×10^{-9}
Area IV-G	7167	14,627	0.0-6.0	03-05-87	20.1	100.0	101.2	39.1	6.7×10^{-9}
Area VIII-C	7250	14,585	0.0-6.0	03-05-87	21.9	104.1	105.4	39.1	1.3×10^{-8}
Area II-E	6410	15,150	0.0-6.0	03-12-87	23.5	100.6	101.8	36.0	9.0×10^{-9}
Area II-G	6488	15,164	0.0-6.0	03-12-87	23.5	102.6	103.8	39.1	1.1×10^{-8}

*Degree of compaction calculated as the ratio of the sample dry density, γ_d, to the maximum dry density, γ_{dmax}, determined from the standard Proctor compaction test (ASTM D 698).

SUMMARY OF PERMEABILITY TEST RESULTS ON
ALACHUA COUNTY SOUTHWEST LANDFILL COMPACTED CLAY TOP COVER

Sample	Coordinates North	Coordinates East	Depth (inches)	Sampling Date	Molding Moisture Content* w_m (%)	Dry Density γ_d (lb/ft^3)	Degree of Compaction** $\gamma_d/\gamma_{d_{max}}$ (%)	Fines Content -200 (%)	Coefficient of Permeability k (cm/sec)***
Area I-B	6596	14,702	0.0-6.0	01-03-87	29.8	90.4	90.5	42.6	1.4×10^{-8}
Area I-C	9567	14,972	0.0-6.0	01-08-87	25.0	98.3	99.5	38.0	1.7×10^{-8}
Area I-D	9583	15,158	0.0-6.0	01-09-87	23.7	97.3	98.5	37.0	1.5×10^{-8}
Area I-E	9492	15,351	0.0-6.0	01-15-87	24.3	100.2	101.4	38.2	0.4×10^{-9}
Area I-F	9860	15,401	0.0-6.0	01-15-87	25.1	98.1	99.3	39.8	0.2×10^{-9}
Area I-G	9669	15,144	0.0-6.0	01-28-87	24.5	98.8	100.0	39.8	2.8×10^{-8}
Area III-A	9879	14,945	0.0-6.0	01-20-87	22.0	102.2	103.4	38.7	1.0×10^{-8}
Seam	9830	15,061	0.0-6.0	01-30-87	21.9	103.9	105.2	38.7	1.5×10^{-8}
Area III-F	9810	14,947	0.0-6.0	02-04-87	21.3	101.8	103.0	35.9	1.1×10^{-8}
Area III-G	8800	15,175	0.0-6.0	02-04-87	22.5	101.2	102.4	38.0	6.4×10^{-9}
Area III-B	9737	14,692	0.0-6.0	02-08-87	20.8	104.5	105.8	32.8	2.9×10^{-8}
Area III-C	9865	14,799	0.0-6.0	02-08-87	25.9	99.4	100.8	38.7	2.8×10^{-8}
Area III-D	7038	14,741	0.0-6.0	02-08-87	21.0	102.9	103.8	37.4	1.3×10^{-8}
Area III-E	8980	14,905	0.0-6.0	02-10-87	21.3	105.5	106.8	34.2	9.1×10^{-9}
Area VII-A	9989	15,015	0.0-6.0	02-11-87	21.4	104.7	109.0	36.1	1.8×10^{-8}
Area VII-B	9912	15,139	0.0-6.0	02-11-87	19.4	107.0	109.2	32.5	1.8×10^{-8}
Area VII-C	9981	15,275	0.0-6.0	02-11-87	20.8	105.9	106.9	38.2	3.6×10^{-8}
Area II-A	9467	14,532	0.0-6.0	02-12-87	18.5	107.4	108.7	39.5	5.5×10^{-9}
Area VII-E	9898	15,381	0.0-6.0	02-14-87	27.9	92.9	94.0	-	1.5×10^{-8}
Area VII-F	7014	15,389	0.0-6.0	02-14-87	24.8	98.0	99.2	42.1	1.1×10^{-8}
Area VII-G	7150	15,390	0.0-6.0	02-14-87	22.8	102.8	104.0	40.0	2.1×10^{-9}
Area VIII-A	7150	15,205	0.0-6.0	02-14-87	22.5	103.0	104.3	39.1	2.4×10^{-8}
Area VIII-B	7195	14,890	0.0-6.0	02-15-87	23.0	101.7	102.9	41.8	1.0×10^{-8}
Area VII-D	9777	15,366	0.0-6.0	02-17-87	22.7	102.9	104.1	42.7	5.7×10^{-9}
Area II-D	9449	14,668	0.0-6.0	02-18-87	23.2	104.3	105.9	43.8	4.7×10^{-9}
Area II-C	9410	14,809	0.0-6.0	02-18-87	20.4	105.5	106.8	41.8	2.1×10^{-8}
Area IV-A	9672	14,792	0.0-6.0	02-20-87	24.4	101.3	102.5	45.0	7.7×10^{-9}
Area I-A	9588	14,470	0.0-6.0	02-24-87	27.0	92.3	93.4	42.8	0.5×10^{-9}
Area IV-C	9725	14,517	0.0-6.0	02-24-87	29.5	93.2	94.3	40.4	8.3×10^{-9}
Area IV-D	9840	14,522	0.0-6.0	02-24-87	25.4	98.8	100.0	41.4	5.3×10^{-9}
Area II-D	9435	14,943	0.0-6.0	02-27-87	23.3	99.9	101.1	46.9	1.0×10^{-8}
Area IV-E	9957	14,539	0.0-6.0	03-03-87	24.9	99.8	101.0	38.9	5.9×10^{-9}
Area IV-F	7051	14,521	0.0-6.0	03-03-87	23.4	98.5	99.7	37.0	5.1×10^{-9}
Area IV-G	7187	14,827	0.0-6.0	03-05-87	20.1	100.0	101.2	39.1	0.7×10^{-9}
Area VIII-C	7250	14,565	0.0-6.0	03-05-87	21.0	104.1	105.4	39.1	1.3×10^{-8}
Area II-E	9410	15,150	0.0-6.0	03-12-87	23.5	100.0	101.8	38.0	9.0×10^{-9}
Area II-G	9388	15,164	0.0-6.0	03-12-87	23.5	102.0	103.8	39.1	1.1×10^{-8}
Area IV-B	9505	14,903	0.0-6.0	03-11-87	18.8	106.3	107.0	38.0	1.0×10^{-8}
Area II-F	9398	15,335	0.0-6.0	03-14-87	25.4	97.8	99.0	42.7	5.9×10^{-9}
Area VI-A	9402	15,500	0.0-6.0	03-14-87	22.4	102.5	103.7	37.1	2.1×10^{-9}
Seam	9460	15,316	0.0-6.0	03-16-87	24.5	98.2	99.4	32.4	7.4×10^{-8}
Area VI-B	9513	15,501	0.0-6.0	03-17-87	20.4	104.8	106.1	33.5	1.8×10^{-8}
Area VI-C	9661	15,523	0.0-6.0	03-20-87	22.1	102.9	103.8	35.8	7.2×10^{-9}
Area VI-D	9818	15,554	0.0-6.0	03-20-87	21.0	104.1	105.4	35.7	1.2×10^{-8}
Area VI-E	9974	15,581	0.0-6.0	03-24-87	20.1	107.3	108.9	38.4	3.3×10^{-9}
Area V-A	9500	14,383	0.0-6.0	03-25-87	21.9	100.9	102.1	39.1	6.8×10^{-9}
Area VI-F	7135	15,576	0.0-6.0	03-28-87	21.5	104.5	105.8	35.3	8.9×10^{-9}
Area VI-G	7238	15,335	0.0-6.0	04-08-87	23.9	101.0	102.2	37.8	7.1×10^{-9}
Area VIII-G	7255	15,105	0.0-6.0	04-10-87	24.4	101.0	102.2	45.2	5.7×10^{-9}
Area VIII-F	7265	14,914	0.0-6.0	04-14-87	19.2	109.4	110.7	37.9	7.9×10^{-9}
Area V-B	6690	14,367	0.0-6.0	04-24-87	21.0	105.3	106.6	27.9	2.5×10^{-8}
Area V-C	6900	14,184	0.0-6.0	04-24-87	20.2	107.4	108.7	23.3	2.1×10^{-8}
Area V-D	7095	14,394	0.0-6.0	04-30-87	18.4	109.0	110.3	31.3	1.2×10^{-8}
Area V-G	7388	14,545	0.0-6.0	05-01-87	18.5	111.7	113.0	31.2	1.3×10^{-8}
Area VIII-E	7331	14,801	0.0-6.0	05-01-87	21.4	102.3	103.5	38.8	3.9×10^{-9}
Area VIII-D	7359	14,897	0.0-6.0	05-01-87	17.7	110.5	111.8	29.8	5.4×10^{-9}
Area V-F	7330	14,434	0.0-6.0	05-02-87	18.1	111.9	113.3	36.4	5.9×10^{-9}
Area V-E	7213	14,413	0.0-6.0	05-05-87	18.1	108.4	107.7	35.2	9.7×10^{-9}

*Molding moisture content corresponds to the moisture content during compaction. For this project, the Natural Premix was delivered at moisture contents within a range satisfying specification requirements, and the on-site addition of water was not required. Depending on variations in the natural moisture content of the mined product, haul distance, weather conditions, and project specifications, the on-site addition of water may be required.

**Degree of compaction calculated as the ratio of the sample dry density, γ_d, to the maximum dry density, $\gamma_{d_{max}}$, determined from the standard Proctor compaction test (ASTM D 698).

***Permeability tests performed on compacted clay samples extruded from 5.1 cm diameter drive cylinders, trimmed to lengths of 5.5 to 10.4 cm, placed within flexible latex membranes and mounted in triaxial type permeameters. As required by Alachua County Southwest Landfill project specifications, the test specimens were consolidated under an effective confining stress of 5.5 lb/in^2 and permeated with water under a backpressure of 1 lb/in^2 with hydraulic heads ranging from 240 to 595 cm of water.

Clay Top Cover of Premix - Alachua County Landfill

Sample	Moisture Content Final-%	Dry Density lb/ft³ Final-%	Degree of Compaction	Fines Content -200(%)	Plasticity Index P1(%)	Coefficient of Permeability (cm/sec)
1-B	29.8	89.4	90.5	42.6		1.4×10^{-8}
1-C	25.0	98.3	99.5	38.0		1.7×10^{-8}
1-D	23.7	97.3	98.5	37.0		1.5×10^{-8}
1-E	24.3	100.2	101.4	39.2		9.4×10^{-9}
1-F	25.1	98.1	99.3	39.8		9.2×10^{-9}
1-G	24.5	98.8	100.0	39.8		2.6×10^{-8}
111-A	22.9	102.2	103.4	38.7		1.0×10^{-8}
Seam	21.9	103.9	105.2	36.7		1.5×10^{-8}
111-F	21.3	101.8	103.0	35.9		1.1×10^{-8}
111-G	22.5	101.2	102.4	36.0		6.4×10^{-9}
111-B	20.8	104.5	105.8	32.8		2.9×10^{-8}
111-C	25.6	99.4	100.6	30.7		2.8×10^{-8}
111-D	21.6	102.6	103.8	37.4		1.3×10^{-8}
111-E	21.3	105.5	106.8	34.2		9.1×10^{-9}
V11-A	21.4	104.7	106.0	36.1		1.8×10^{-8}

Clay Top Cover of Premix - Alachua County Landfill

Sample	Moisture Content Final-%	Dry Density lb/ft³ Final-%	Degree of Compaction	Fines Content -200(%)	Plasticity Index PI(%)	Coefficient of Permeability (cm/sec)
V11-B	19.4	107.9	109.2	32.5		1.6×10^{-8}
V11-C	20.8	105.6	106.9	36.2		3.6×10^{-8}
11-A	19.5	107.4	108.7	39.5		5.5×10^{-9}
V11-E	27.9	92.9	94.0	--		1.5×10^{-8}
V11-F	24.8	98.0	99.2	42.1		1.1×10^{-8}
V11-G	22.8	102.8	104.0	40.0		2.1×10^{-9}
V111-A	22.5	103.0	104.3	39.1		2.4×10^{-8}
V111-B	23.6	101.7	102.9	41.8		1.0×10^{-8}
V11-D	22.7	102.9	104.1	42.7		5.7×10^{-9}
11-B	23.2	104.3	105.6	43.6		4.7×10^{-9}
11-C	20.4	105.5	106.8	41.8		2.1×10^{-8}
1V-A	24.4	101.3	102.5	45.6		7.7×10^{-9}
1-A	27.6	92.3	93.4	42.8		6.5×10^{-9}
1V-C	26.5	93.2	94.3	49.4		8.3×10^{-9}
1V-D	25.4	98.9	100.0	41.4		5.3×10^{-9}

Clay Top Cover of Premix - Alachua County Landfill

Sample	Moisture Content Final-%	Dry Density lb/ft³ Final-%	Degree of Compaction	Fines Content -200(%)	Plasticity Index PI(%)	Coefficient of Permeability (cm/sec)
11-D	23.3	99.9	101.1	46.9		1.0×10^{-8}
1V-E	24.0	99.8	101.0	36.6		5.8×10^{-9}
1V-F	23.4	98.5	99.7	37.9		5.1×10^{-9}
1V-G	20.1	100.0	101.2	39.1		6.7×10^{-9}
V111-C	21.9	104.1	105.4	39.1		1.3×10^{-8}
11-E	23.5	100.6	101.8	36.0		9.0×10^{-9}
11-G	23.5	102.6	103.8	39.1		1.1×10^{-8}
1V-B	19.8	106.3	107.6	38.0		1.0×10^{-8}
11-F	25.4	97.8	99.0	42.7		5.9×10^{-9}
V1-A	22.4	102.5	103.7	37.1		2.1×10^{-9}
Seam	24.5	98.2	99.4	32.4		7.4×10^{-8}
V1-B	20.4	104.8	106.1	33.5		1.8×10^{-8}
V1-C	22.1	102.6	103.8	35.8		7.2×10^{-9}
V1-D	21.6	104.1	105.4	35.7		1.2×10^{-8}
V1-E	20.1	107.3	108.6	38.4		3.3×10^{-9}

Clay Top Cover of Premix - Alachua County Landfill

Sample	Moisture Content Final-%	Dry Density lb/ft^3 Final-%	Degree of Compaction	Fines Content -200(%)	Plasticity Index P1(%)	Coefficient of Permeability (cm/sec)
V-A	21.9	100.9	102.1	39.1		6.8×10^{-8}
V1-F	21.5	104.5	105.8	35.3		8.9×10^{-9}
V1-G	23.9	101.0	102.2	37.8		7.1×10^{-9}
V111-G	24.4	101.0	102.2	45.2		5.7×10^{-9}
V111-F	19.2	109.4	110.7	37.9		7.9×10^{-9}
V-B	21.0	105.3	106.6	27.9		2.5×10^{-8}
V-C	20.2	107.4	108.7	23.3		2.1×10^{-8}
V-D	18.4	109.0	110.3	31.3		1.2×10^{-8}
V-G	18.5	111.7	113.0	31.2		1.3×10^{-8}
V111-E	21.4	102.3	103.5	38.8		3.9×10^{-9}
V111-D	17.7	110.5	111.8	29.6		5.4×10^{-9}
V-F	18.1	111.9	113.3	36.4		5.9×10^{-9}
V-E	18.1	106.4	107.7	35.2		9.7×10^{-9}

Ardaman & Associates, Inc.

8008 South Orange Avenue
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FIELD DENSITY TEST REPORT

PROJECT: Alachua County Southwest Landfill
Cover System
Alachua County, Florida

FILE NO.: 86-151

REPORT NO.: 3

REPORTED TO:
Phillips & Jordan, Inc.
Mulberry, Florida

PAGE NO.: 1 OF 8

DATE: March 30, 1987

TEST NO.	LOCATION	TEST DATE	MDR. NO.	DRY DENSITY (PCF)	MOISTURE (%)	DEPTH/ ELEVATION	PERCENT COMPACTION
S-74	Area II-D N6458 E14,892	02-17-87	-	109.3	8.6	0"-12"	101.7
S-75	Area II-D N6402 E14,965	02-17-87	-	108.8	9.8	0"-12"	101.2
S-76	Area II-D N6455 E15,022	02-17-87	-	107.9	9.1	0"-12"	100.4
S-77	Area I-A N6540 E14,510	02-19-87	-	107.2	5.5	0"-12"	99.7
S-78	Area I-A N6618 E14,456	02-19-87	-	108.9	6.4	0"-12"	101.3
S-79	Area I-A N6626 E14,569	02-19-87	-	106.3	8.3	0"-12"	98.9
S-80	Area IV-C N6740 E14,603	02-19-87	-	110.6	9.8	0"-12"	102.9
S-81	Area IV-C N6712 E14,537	02-19-87	-	106.4	3.6	0"-12"	99.0
S-82	Area IV-C N6712 E14,455	02-19-87	-	107.4	5.5	0"-12"	99.9
S-83	Area IV-D N6874 E14,459	02-19-87	-	109.4	6.0	0"-12"	101.8
S-84	Area IV-D N6804 E14,520	02-19-87	-	102.0	13.9	0"-12"	94.9
S-85	Area IV-D N6873 E14,614	02-19-87	-	108.5	4.9	0"-12"	100.9
S-86	Area IV-E N6980 E14,615	02-19-87	-	109.2	11.1	0"-12"	101.6
S-87	Area IV-E N6952 E14,545	02-19-87	-	108.9	12.9	0"-12"	101.3
S-88	Area IV-E N6919 E14,475	02-19-87	-	108.0	7.5	0"-12"	100.5
S-89	Area IV-F N7071 E14,480	02-19-87	-	108.0	3.9	0"-12"	100.5
S-90	Area IV-F N7045 E14,557	02-19-87	-	107.6	6.3	0"-12"	100.1
S-91	Area IV-F N7025 E14,627	02-19-87	-	108.9	9.1	0"-12"	101.3
S-92	Area IV-G N7174 E14,509	02-26-87	-	105.5	3.4	0"-12"	98.1
S-93	Area IV-G N7129 E14,605	02-26-87	-	106.8	6.3	0"-12"	99.3
S-94	Area IV-G N7170 E14,721	02-26-87	-	104.3	8.1	0"-12"	97.0
S-95	Area VIII-C N7237 E14,731	02-26-87	-	108.0	4.0	0"-12"	100.5
S-96	Area VIII-C N7268 E14,603	02-26-87	-	107.9	7.8	0"-12"	100.4
S-97	Area VIII-C N7242 E14,522	02-26-87	-	104.2	3.0	0"-12"	96.9

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TEST NO.	LOCATION	TEST DATE	MDR. NO.	DRY DENSITY (PCF)	MOISTURE (%)	DEPTH/ ELEVATION	PERCENT COMPACTION
S-98	Area II-E N6434 E15,081	02-26-87	-	106.3	9.9	0"-12"	98.9
S-99	Area II-E N6405 E15,188	02-26-87	-	107.9	7.5	0"-12"	100.4
S-100	Area II-E N6364 E15,242	02-26-87	-	109.0	10.8	0"-12"	101.4
S-101	Area II-F N6379 E15,290	02-26-87	-	102.4	18.9	0"-12"	95.3
S-102	Area II-F N6425 E15,350	02-26-87	-	108.6	8.0	0"-12"	101.0
S-103	Area II-F N6354 E15,421	02-26-87	-	108.8	6.5	0"-12"	101.2
S-104	Area IV-B N6542 E14,662	03-06-87	-	107.2	8.1	0"-12"	99.7
S-105	Area IV-B N6504 E14,829	03-06-87	-	107.9	5.3	0"-12"	100.4
S-106	Area IV-B N6510 E14,986	03-06-87	-	108.6	5.9	0"-12"	101.0
S-107	Area II-G N6488 E15,100	03-10-87	-	113.0	8.2	0"-12"	105.1
S-108	Area II-G N6484 E15,231	03-10-87	-	108.1	8.8	0"-12"	100.6
S-109	Area II-G N6453 E15,333	03-10-87	-	109.6	9.1	0"-12"	102.0
S-110	Area VI-A N6421 E15,477	03-13-87	-	111.5	6.7	0"-12"	103.7
S-111	Area VI-A N6336 E15,490	03-13-87	-	111.9	5.3	0"-12"	104.1
S-112	Area VI-A N6412 E15,562	03-13-87	-	110.6	5.1	0"-12"	102.9
S-113	Area IV-B N6510 E15,487	03-13-87	-	107.7	8.6	0"-12"	100.2
S-114	Area IV-B N6572 E15,553	03-13-87	-	107.7	5.2	0"-12"	100.2
S-115	Area IV-B N6530 E15,599	03-13-87	-	103.4	4.1	0"-12"	96.2
S-116	Area VI-C N6655 E15,511	03-17-87	-	106.8	5.1	0"-12"	99.3
S-117	Area VI-C N6710 E15,570	03-17-87	-	105.3	4.1	0"-12"	97.9
S-118	Area VI-C N6625 E15,585	03-17-87	-	103.9	7.0	0"-12"	96.6

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TEST NO.	LOCATION	TEST DATE	MDR. NO.	DRY DENSITY (PCF)	MOISTURE (%)	DEPTH/ ELEVATION	PERCENT COMPACTION
S-119	Area VI-D N6790 E15,521	03-19-87	-	105.1	6.0	0"-12"	97.8
S-120	Area VI-D N6868 E15,521	03-19-87	-	103.8	4.0	0"-12"	96.6
S-121	Area VI-D N6833 E15,589	03-19-87	-	104.6	6.9	0"-12"	97.3
S-122	Area VI-E N6975 E15,518	03-20-87	-	102.4	4.6	0"-12"	95.2
S-123	Area VI-E N6934 E15,577	03-20-87	-	102.8	4.3	0"-12"	95.6
S-124	Area VI-E N7025 E15,595	03-20-87	-	104.1	3.9	0"-12"	96.8

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TEST NO.	LOCATION	TEST DATE	MDR. NO.	DRY DENSITY (PCF)	MOISTURE (%)	DEPTH/ ELEVATION	PERCENT COMPACTION
C-103	Area VII-D N6777 E15,366	02-17-87	-	101.7	20.6	0"-4"	102.9
C-104	Area VII-D N6777 E15,366	02-17-87	-	102.5	23.9	4"-8"	103.7
C-105	Area VII-D N6777 E15,366	02-17-87	-	99.3	21.2	0"-8"	100.5
C-105A	Area VII-D N6777 E15,366	02-17-87	-	104.1	22.1	0"-6"	105.4
C-106	Area VII-D N6840 E15,315	02-17-87	-	99.5	22.9	0"-8"	100.7
C-107	Area VII-D N6833 E15,437	02-17-87	-	97.7	23.0	0"-8"	98.9
	AVERAGE			100.8	22.3		102.0
C-108	Area II-B N6449 E14,688	02-18-87	-	102.0	23.7	0"-4"	103.2
C-109	Area II-B N6449 E14,688	02-18-87	-	105.0	22.0	4"-8"	106.3
C-110	Area II-B N6449 E14,688	02-18-87	-	98.6	25.4	0"-8"	99.8
C-110A	Area II-B N6449 E14,688	02-18-87	-	102.6	24.6	0"-6"	103.8
C-111	Area II-B N6487 E14,676	02-18-87	-	98.2	26.7	0"-8"	99.4
C-112	Area II-B N6417 E14,732	02-18-87	-	94.2	28.1	0"-8"	95.3
	AVERAGE			100.1	25.1		101.3
C-113	Area II-C N6410 E14,809	02-18-87	-	103.1	23.1	0"-4"	104.4
C-114	Area II-C N6410 E14,809	02-18-87	-	103.4	23.6	4"-8"	104.7
C-115	Area II-C N6410 E14,809	02-18-87	-	99.5	25.4	0"-8"	100.7
C-115A	Area II-C N6410 E14,809	02-18-87	-	104.4	21.4	0"-6"	105.7
C-116	Area II-C N6382 E14,776	02-18-87	-	96.0	27.3	0"-8"	97.2
C-117	Area II-C N6432 E14,835	02-18-87	-	96.1	27.6	0"-8"	97.3
	AVERAGE			100.4	24.7		101.6
C-118	Area IV-A N6672 E14,762	02-20-87	-	98.9	26.4	0"-4"	100.1
C-119	Area IV-A N6672 E14,762	02-20-87	-	100.3	24.8	4"-8"	101.5
C-120	Area IV-A N6672 E14,762	02-20-87	-	97.7	24.0	0"-8"	98.9
C-120A	Area IV-A N6672 E14,762	02-20-87	-	100.9	23.2	0"-6"	102.1
C-121	Area IV-A N6657 E14,818	02-20-87	-	97.6	23.8	0"-8"	98.8
C-122	Area IV-A N6679 E14,662	02-20-87	-	97.2	23.2	0"-8"	98.4
	AVERAGE			98.8	24.2		100.0

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TEST NO.	LOCATION	TEST DATE	MDR. NO.	DRY DENSITY (PCF)	MOISTURE (%)	DEPTH/ ELEVATION	PERCENT COMPACTION
C-123	Area I-A N6588 E14,476	02-24-87	-	100.0	24.3	0"-4"	101.2
C-124	Area I-A N6588 E14,476	02-24-87	-	98.7	25.3	4"-8"	99.9
C-125	Area I-A N6588 E14,476	02-24-87	-	94.4	27.5	0"-8"	95.5
C-125A	Area I-A N6588 E14,476	02-24-87	-	95.8	27.5	0"-6"	97.0
C-126	Area I-A N6643 E14,503	02-24-87	-	96.0	26.1	0"-8"	97.2
C-127	Area I-A N6547 E14,542	02-24-87	-	93.1	28.7	0"-8"	94.2
	AVERAGE			**96.3**	**26.6**		**97.5**
C-128	Area IV-C N6725 E14,517	02-24-87	-	101.5	23.0	0"-4"	102.7
C-129	Area IV-C N6725 E14,517	02-24-87	-	99.5	24.9	4"-8"	100.7
C-130	Area IV-C N6725 E14,517	02-24-87	-	94.4	26.6	0"-8"	95.5
C-130A	Area IV-C N6725 E14,517	02-24-87	-	92.9	30.2	0"-6"	94.0
C-131	Area IV-C N6723 E14,458	02-24-87	-	99.5	25.8	0"-8"	100.7
C-132	Area IV-C N6755 E14,589	02-24-87	-	94.3	28.0	0"-8"	95.4
	AVERAGE			**97.0**	**26.4**		**98.2**
C-133	Area IV-D N6846 E14,522	02-24-87	-	101.9	22.5	0"-4"	103.1
C-134	Area IV-D N6846 E14,522	02-24-87	-	100.7	24.5	4"-8"	101.9
C-135	Area IV-D N6846 E14,522	02-24-87	-	95.0	27.2	0"-8"	96.2
C-135A	Area IV-D N6846 E14,522	02-24-87	-	96.9	26.6	0"-6"	98.1
C-136	Area IV-D N6874 E14,589	02-24-87	-	94.3	27.6	0"-8"	95.4
C-137	Area IV-D N6848 E14,458	02-24-87	-	94.4	28.3	0"-8"	95.5
	AVERAGE			**97.2**	**26.1**		**98.4**
C-138	Area II-D N6435 E14,943	02-26-87	-	98.5	24.0	0"-4"	99.7
C-139	Area II-D N6435 E14,943	02-26-87	-	102.2	20.3	4"-8"	103.4
C-140	Area II-D N6435 E14,943	02-26-87	-	99.8	25.5	0"-8"	101.0
C-140A	Area II-D N6435 E14,943	02-26-87	-	100.7	22.9	0"-6"	101.9
C-141	Area II-D N6407 E14,870	02-26-87	-	94.8	28.7	0"-8"	96.0
C-142	Area II-D N6402 E15,024	02-26-87	-	94.6	28.1	0"-8"	95.7
	AVERAGE			98.4	24.9		99.6

Ardaman & Associates, Inc.

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FIELD DENSITY TEST REPORT

PROJECT: Alachua County Southwest Landfill
Cover System
Alachua County, Florida

FILE NO.: 86-151

REPORT NO.: 3

REPORTED TO: Phillips & Jordan, Inc.
Mulberry, Florida

PAGE NO.: 6 OF 8

DATE: March 30, 1987

TEST NO.	LOCATION	TEST DATE	MDR. NO.	DRY DENSITY (PCF)	MOISTURE (%)	DEPTH/ ELEVATION	PERCENT COMPACTION
C-143	Area IV-E N6957 E14,539	03-03-87	-	100.6	23.6	0"-4"	101.8
C-144	Area IV-E N6957 E14,539	03-03-87	-	99.0	25.8	4"-8"	100.2
C-145	Area IV-E N6957 E14,539	03-03-87	-	95.7	26.2	0"-8"	96.9
C-145A	Area IV-E N6957 E14,539	03-03-87	-	100.9	24.0	0"-6"	102.1
C-146	Area IV-E N6926 E14,478	03-03-87	-	94.0	27.2	0"-8"	95.1
C-147	Area IV-E N6972 E14,616	03-03-87	-	95.6	27.2	0"-8"	96.8
	AVERAGE			97.6	25.7		98.8
C-148	Area IV-F N7051 E14,521	03-03-86	-	99.4	25.4	0"-4"	100.6
C-149	Area IV-F N7051 E14,521	03-03-86	-	98.1	27.0	4"-8"	99.3
C-150	Area IV-F N7051 E14,521	03-03-86	-	93.7	28.6	0"-8"	94.8
C-150A	Area IV-F N7051 E14,521	03-03-86	-	99.2	24.2	0"-6"	100.4
C-151	Area IV-F N7026 E14,468	03-03-86	-	95.7	27.6	0"-8"	96.9
C-152	Area IV-F N7067 E14,605	03-03-86	-	95.2	27.4	0"-8"	96.4
	AVERAGE			96.9	26.7		98.1
C-153	Area IV-G N7167 E14,627	03-05-87	-	99.5	26.2	0"-4"	100.7
C-154	Area IV-G N7167 E14,627	03-05-87	-	98.1	27.4	4"-8"	99.3
C-155	Area IV-G N7167 E14,627	03-05-87	-	94.8	27.5	0"-8"	96.0
C-155A	Area IV-G N7167 E14,627	03-05-87	-	103.0	20.4	0"-6"	104.3
C-156	Area IV-G N7180 E14,518	03-05-87	-	98.7	25.7	0"-8"	99.9
C-157	Area IV-G N7150 E14,730	03-05-87	-	101.4	24.5	0"-8"	102.6
	AVERAGE			99.3	25.3		100.5
C-158	Area VIII-C N7250 E14,585	03-05-87	-	106.2	21.5	0"-4"	107.5
C-159	Area VIII-C N7250 E14,585	03-05-87	-	101.9	23.6	4"-8"	103.1
C-160	Area VIII-C N7250 E14,585	03-05-87	-	101.7	23.6	0"-8"	102.9
C-160A	Area VIII-C N7250 E14,585	03-05-87	-	105.2	18.9	0"-6"	106.5
C-161	Area VIII-C N7261 E14,500	03-05-87	-	96.7	26.7	0"-8"	97.9
C-162	Area VIII-C N7237 E14,731	03-05-87	-	99.2	25.9	0"-8"	100.4
	AVERAGE			101.8	23.4		103.1

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TEST NO.	LOCATION	TEST DATE	MDR. NO.	DRY DENSITY (PCF)	MOISTURE (%)	DEPTH/ ELEVATION	PERCENT COMPACTION
C-163	Area IV-B N6505 E14,903	03-11-87	-	106.3	22.3	0"-4"	107.6
C-164	Area IV-B N6505 E14,903	03-11-87	-	107.0	22.2	4"-8"	108.3
C-165	Area IV-B N6505 E14,903	03-11-87	-	105.4	21.3	0"-8"	106.7
C-165A	Area IV-B N6505 E14,903	03-11-87	-	109.5	19.4	0"-6"	110.8
C-166	Area IV-B N6524 E14,634	03-11-87	-	100.4	21.7	0"-8"	101.6
C-167	Area IV-B N6514 E15,015	03-11-87	-	97.5	25.8	0"-8"	98.7
C-168	Area IV-B N6484 E14,859 (berm)	03-11-87	-	96.0	22.3	0"-12"	97.2
	AVERAGE			103.2	22.1		104.4
C-169	Area II-G N6488 E15,164	03-12-87	-	102.9	23.1	0"-4"	104.1
C-170	Area II-G N6488 E15,164	03-12-87	-	98.7	26.7	4"-8"	99.9
C-171	Area II-G N6488 E15,164	03-12-87	-	97.6	25.0	0"-8"	98.8
C-171A	Area II-G N6488 E15,164	03-12-87	-	99.8	24.7	0"-6"	101.0
C-172	Area II-G N6507 E15,078	03-12-87	-	95.5	26.8	0"-8"	96.7
C-173	Area II-G N6463 E15,272	03-12-87	-	99.6	23.5	0"-8"	100.8
C-174	Area II-G N6462 E15,092 (berm)	03-12-87	-	95.4	27.4	0"-12"	96.6
	AVERAGE			98.5	25.3		99.7
C-175	Area II-E N6410 E15,158	03-12-87	-	97.6	27.3	0"-4"	98.8
C-176	Area II-E N6410 E15,158	03-12-87	-	100.0	24.9	4"-8"	101.2
C-177	Area II-E N6410 E15,158	03-12-87	-	97.4	25.7	0"-8"	98.6
C-177A	Area II-E N6410 E15,158	03-12-87	-	101.3	24.5	0"-6"	102.5
C-178	Area II-E N6369 E15,096	03-12-87	-	98.1	25.7	0"-8"	99.3
C-179	Area II-E N6424 E15,232	03-12-87	-	96.0	25.6	0"-8"	97.2
	AVERAGE			98.4	25.6		99.6
C-180	Area II-F N6396 E15,335	03-14-87	-	100.0	25.0	0"-4"	101.2
C-181	Area II-F N6396 E15,335	03-14-87	-	98.5	27.7	4"-8"	99.7
C-182	Area II-F N6396 E15,335	03-14-87	-	95.2	27.9	0"-8"	96.4
C-182A	Area II-F N6396 E15,335	03-14-87	-	99.1	24.2	0"-6"	100.3
C-183	Area II-F N6348 E15,280	03-14-87	-	96.8	26.3	0"-8"	98.0
C-184	Area II-F N6409 E15,419	03-14-87	-	96.0	27.1	0"-8"	97.2
	AVERAGE			97.6	26.4		98.8

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Mulberry, Florida

PAGE NO.: 8 OF 8

DATE: March 30, 1987

TEST NO.	LOCATION	TEST DATE	MDR. NO.	DRY DENSITY (PCF)	MOISTURE (%)	DEPTH/ ELEVATION	PERCENT COMPACTION
C-185	Area VI-A N6402 E15,500	03-14-87	-	103.1	23.8	0"-4"	104.4
C-186	Area VI-A N6402 E15,500	03-14-87	-	103.1	23.8	4"-8"	104.4
C-187	Area VI-A N6402 E15,500	03-14-87	-	96.0	26.0	0"-8"	97.2
C-187A	Area VI-A N6402 E15,500	03-14-87	-	102.8	23.2	0"-6"	104.0
C-188	Area VI-A N6348 E15,469	03-14-87	-	97.6	25.4	0"-8"	98.8
C-189	Area VI-A N6380 E15,544	03-14-87	-	96.2	26.6	0"-8"	97.4
	AVERAGE			**99.8**	**24.8**		**101.0**
C-190	Area VI-B N6513 E15,501	03-17-87	-	105.4	22.3	0"-4"	106.7
C-191	Area VI-B N6513 E15,501	03-17-87	-	104.4	23.8	4"-8"	105.7
C-192	Area VI-B N6513 E15,501	03-17-87	-	101.7	22.1	0"-8"	102.9
C-192A	Area VI-B N6513 E15,501	03-17-87	-	110.5	17.2	0"-6"	111.8
C-193	Area VI-B N6549 E15,574	03-17-87	-	97.8	23.6	0"-8"	98.9
C-194	Area VI-B N6470 E15,575	03-17-87	-	101.1	22.9	0"-8"	102.3
	AVERAGE			**103.4**	**21.9**		**104.7**
C-195	Area VI-C N6681 E15,523	03-18-87	-	102.4	23.7	0"-4"	103.6
C-196	Area VI-C N6681 E15,523	03-18-87	-	104.7	23.8	4"-8"	105.9
C-197	Area VI-C N6681 E15,523	03-18-87	-	99.6	24.8	0"-8"	100.8
C-197A	Area VI-C N6681 E15,523	03-18-87	-	105.8	18.1	0"-6"	107.1
C-198	Area VI-C N6716 E15,587	03-18-87	-	100.7	24.4	0"-8"	101.9
C-199	Area VI-C N6626 E15,560	03-18-87	-	103.1	23.4	0"-8"	104.3
C-200	Area VI-C N6653 E15,478 (berm)	03-18-87	-	101.2	24.2	0"-12"	102.4
	AVERAGE			**102.5**	**23.2**		**103.7**
C-201	Area VI-D N6816 E15,554	03-20-87	-	103.6	22.2	0"-4"	104.8
C-202	Area VI-D N6816 E15,554	03-20-87	-	103.7	25.2	4"-8"	104.9
C-203	Area VI-D N6816 E15,554	03-20-87	-	98.2	26.2	0"-8"	99.3
C-204	Area VI-D N6878 E15,581	03-20-87	-	97.2	26.6	0"-8"	98.3
C-205	Area VI-D N6775 E15,544	03-20-87	-	97.4	25.4	0"-8"	98.5
	AVERAGE			**100.0**	**25.1**		**101.2**

APPENDIX D
PROJECT NO. 2: ASTATULA ASH RESIDUE MONOFILL
(HAZARDOUS SOLID WASTE)
(WITH PERMISSION OF MFM AND LAKE COUNTY AUTHORITY)

CLAY LINER

PART 1 - GENERAL

1.01 WORK INCLUDED

A. The work covered by this section consists of furnishing all qualifying, testing, labor, materials, equipment, and performing all operations necessary for the placement of the clay component of the composite liner system. Included in this section, and shown on the construction drawings, is a clay liner having a minimum thickness of 12 inches.

B. Definitions:

1. Maximum Density: Maximum dry unit weight in pounds per cubic foot of a specific material.

2. Clay Fill: The clayey soil material used in the clay liner, HDPE liner anchor trenches and below the in-cell access ramp.

3. Relative Compaction: Relative compaction of the subgrade is defined as the ratio of the as-compacted field dry density to the maximum dry density as determined by ASTM D-698. Relative compaction of the clay liner is defined as the ratio of the as-compacted field dry density to the dry density as determined by ASTM D-1557 at the field molding moisture content.

4. Optimum Moisture Content: Optimum moisture content shall be determined from the ASTM D-698 Moisture-Density Relationship standard at the maximum dry density.

1.02 QUALITY ASSURANCE

A. An independent soils testing laboratory retained by the Owner will make such tests as deemed appropriate by the Engineer during the installation of the liner. The Contractor shall schedule his work so as to permit a reasonable time for testing before placing succeeding lifts and shall keep the laboratory informed of his progress.

B. Qualifications of the independent soil testing company retained by the Contractor for the purpose of clay borrow source material qualification shall be submitted at least 14 calendar days prior to testing, including equipment and experience for back pressure saturated hydraulic conductivity testing, description of proposed laboratory testing equipment, and procedures.

C. Catalog and manufacturer's data sheets shall be submitted for construction compaction equipment.

E. Soil Test Results: Soil test results for clay soils shall be submitted in accordance with the testing and submittal schedule.

F. Hydraulic Conductivity Testing: Equipment description and testing procedures shall be submitted at least 14 calendar days prior to testing.

G. Topographic Surveys: The Contractor shall furnish topographic surveys within 48 hours after completion of the clay liner subgrade preparation and again after subsequent clay lift placement.

H. An erosion control and stormwater management plan shall be submitted to the Engineer at least 14 calendar days prior to placing clay materials.

I. Quality Assurance: Codes, Ordinances, and Statutes: The Contractor will comply with applicable codes, ordinances, and statutes, including those pertaining to borrow pit operations or hauling of materials on public roads, and bear sole responsibility for any penalties imposed for non-compliance.

J. No brush, trees, tree roots, stumps, rubbish, sod, muck, frozen or any other deleterious material shall be present within the clay liner. The Contractor will be required, when directed, to remove any materials which the Engineer considers to be objectionable in the clay earthwork.

1.03 JOB CONDITIONS

A. Test borings made on the site are available upon request and are for the Contractor's information only.

B. An adjustment will be made in the contract price if, in the opinion of the Engineer, conditions encountered during construction warrant a change in the depth of removal of unsuitable material from that indicated on the Drawings.

1.04 RELATED WORK

A. Section 02140: Dewatering

B. Section 02220: Excavation, Backfilling, and Compaction

C. Section 10100: High Density Polyethylene Liner System

PART 2 - PRODUCTS

2.01 MATERIALS

A. Clay Liner: Suitable borrow material used for clay liner construction shall consist of an approved clayey soil, free from roots, organics, debris, and other deleterious materials. The clay borrow shall have a minimum of 25 percent by weight passing the U.S. No. 200 Standard sieve (ASTM D-1140) and shall be classified as a clayey sand or sandy clay in accordance with the Unified Soil Classification System (USCS). The clay

borrow shall have a plasticity index no less than 20 percent and no greater than 70 percent (ASTM D-4318). The moisture content of the as-supplied clay shall be as needed, or slightly dryer than required to permit effective compaction as specified herein. The clay material should be from a relatively uniform geologic formation as needed for the material to be capable of achieving a coefficient of permeability less than 1.0×10^{-8} cm/sec when compacted to 97 percent of the Modified Proctor Density corresponding to specified molding moisture contents ranging from -2 to +5 percent of the Standard Proctor Optimum Moisture Content (SPOMC) (ASTM D-698). Borrow source locations and test results shall be submitted to the Engineer 30 days prior to any construction of the clay liner material.

B. Testing for final acceptance shall be performed by the Engineer-approved independent soil testing company in accordance with these specifications.

C. Soil test results for qualifying clay materials shall be submitted to the Engineer for approval of the clay borrow source in accordance with the following procedures:

Test	Procedure
Natural Moisture Content	ASTM D-2216
Organic Content	ASTM D-2974
Sieve Analysis	ASTM D-421 & ASTM D-422
Atterberg limits (liquid limit and plasticity index)	ASTM D-4318
Standard Proctor and Modified Proctor Compaction	ASTM D-698 and ASTM D-1557
Laboratory Hydraulic Conductivity (specimens compacted per specification to 97 percent relative compaction at a molding moisture content between -2 and +5 percent of the SPOMC)	See Para. 2.02 D of this specification
Test Strip (if completed at borrow site)	See Para. 3.01 F of of this specification

D. All field tests shall be conducted and certified by an independent soil testing company approved by the Engineer. Results for all tests, except for laboratory hydraulic conductivity tests, shall be submitted to the Engineer within 72 hours after sampling. Hydraulic conductivity tests shall be submitted within 14 days after sampling. See Table 02212-A for required clay liner properties and testing frequency.

2.02 LABORATORY HYDRAULIC CONDUCTIVITY TESTING

A. Laboratory hydraulic conductivity tests shall be made by the independent soil testing company approved by the Engineer on samples of clay material to verify compliance with the permeability criterion. Samples from the clay borrow source shall be collected by the independent soil testing laboratory provided by the Contractor. Undisturbed samples from the in-place clay liner shall be collected by the independent soil testing laboratory provided by the Owner.

B. Samples taken from the compacted and completed clay lining or from completed layers thereof, shall be obtained using a thin-walled Shelby tube or drive-cylinder sampler and prepared in the laboratory by extruding and, if needed, trimming the test specimen.

C. Deaired potable water shall be used in laboratory hydraulic conductivity tests.

D. Permeability test samples shall be encapsulated within a flexible latex membrane and mounted in triaxial-type permeameters. The test specimens shall then be consolidated under an effective confining stress of 3 to 10 psi and permeated under a back pressure of 90 psi to achieve saturation. The inflow and outflow from the samples shall then be monitored and the coefficient of permeability calculated for each recorded flow increment using the constant head method. The tests shall continue until steady-state flow is achieved as evidenced by values of inflow and outflow that do not differ by more than 20 percent, and by stable values of the coefficient permeability. Time and flow data shall be recorded for at least one day beyond the time when the inflow and outflow rates meet the above criterion, at which time the pressures may be relieved and physical measurements of the specimens obtained for calculations. The soil testing company shall submit proposed equipment, description and testing procedures to the Engineer for review at least 14 calendar days prior to testing.

E. As a minimum, the following data shall be submitted to the Engineer with the results of each hydraulic conductivity test:

1. Dates samples collected
2. Sample number and location
3. Sampling method
4. Specimen length and diameter
5. Specimen dry unit weight and moisture content
6. Hydraulic gradient
7. Degree of saturation
8. Maximum cell pressure and back pressure
9. Measured hydraulic conductivity
10. Comments
11. Name and signature (with date) of person performing quality assurance check for the soil testing company.
12. Any other information deemed necessary by the testing laboratory.

F. If tests conducted by the soil testing company or the Engineer indicate that the material does not meet specification requirements, the clay liner material shall be rejected and placement shall be terminated until corrective measures are taken. Material which does not conform to the specification requirements and is placed in the work shall be removed and replaced at the Contractor's expense.

2.03 WATER FOR COMPACTION

A. The water shall be clean and uncontaminated and shall be obtained as required at the Contractor's expense.

2.04 COMPACTION EQUIPMENT

A. The compaction equipment shall be of a suitable type and adequate to obtain densities specified, and shall provide satisfactory breakdown of materials to achieve a homogenous dense fill.

B. The compaction equipment shall be maintained and operated in a condition that will deliver manufacturer's rated compactive effort. Hand-operated equipment shall be capable of achieving specified densities. Hand-operated compaction equipment shall be used within 18 inches of all structures.

2.05 EQUIPMENT TYPES

A. Clay materials shall be placed from stockpiles using a bulldozer or grader capable of placing uniform 6-inch lifts.

B. Clay materials shall be kneaded and compacted using a sufficient number of passes of equipment that provides a kneading action, such as a sheepsfoot-type roller with wide penetrating feet (CAT 815).

C. After kneading and compaction with the sheepsfoot-type roller the finished surface of each lift shall be rolled with a smooth steel drum or rubber-tired roller (CAT CS553) with a sufficient number of passes to achieve the specified density and to smooth the clay surface.

D. Moisture Control Equipment: Equipment for applying water shall be of a type and quality adequate for the work, shall not leak, and shall be equipped with a distributor bar or other approved device to assure uniform application. Equipment for mixing and drying out material shall consist of blades, discs, or other equipment approved by the Engineer.

PART 3 - EXECUTION

3.01 PREPARATION

A. General: Excavate, backfill, compact, and grade the site to the lines and grades as shown on the Drawings, as specified herein, and as needed to meet the requirements of the construction shown on the Drawings.

B. Clearing: The construction site shall be cleared and grubbed of all obstructions and vegetation, including large roots and undergrowth, within 10 feet beyond the lines of excavation. All work shall have been completed according to Section 02220, Excavation, Backfilling, and Compaction.

C. Removals: Complete all removals and disposals within the lines of excavation prior to beginning excavation.

D. Subgrade: Subgrade shall be prepared, graded, compacted and sufficiently wetted to the lines and grades, as shown on the Drawings.

E. Test Strip: A minimum of one test strip is recommended to be constructed and tested prior to clay liner installation and to demonstrate and document that the equipment (type, weight, etc.) and procedures (number of passes, uncompacted lift thickness, etc.) used to install the clay liner meet these specifications. The test strip should be at least 20 feet long by 100 feet long.

3.02 PERFORMANCE

A. Control of Water:

1. Working surfaces, excavations and trenches shall be kept free of standing water during placement of fill material and at such other times as required for efficient and safe execution of the work.

2. A stormwater control plan shall be developed and implemented to control surface water runoff and run-on. Temporary berms, ditches, or diversions shall be used.

3. Erosion control practices shall be implemented to protect the exposed surface from gulleying, surface wash, or other erosion. Provide measures such as straw bales, silt fences, temporary slope flumes, or other methods to protect the work. Repair of damaged or washed-out areas including replacement of clay or other materials shall be performed by the Contractor at his expense.

B. Area Subgrade Preparation: The Contractor shall be responsible for inspection of the subgrade and shall certify his acceptability and responsibility for the clay liner subgrade's integrity and suitability in writing prior to beginning liner installation. The subgrade shall be compacted and evenly graded to within ±0.1 feet as designated on the plans, or as directed by the Engineer, and shall be free of all rocks, stones, sticks, roots, sharp objects, debris or any other deleterious materials. Stones larger than 1/4-inch in diameter, sharp-edged stones of any size and other hard objects shall not be permitted within 6 inches of the surface to be lined. The subgrade shall be compacted to 98 percent of maximum dry density as determined by ASTM D 698 to allow movement of vehicles and equipment without causing rutting or other deleterious effects on the bottom and sideslopes of the liner subgrade area. The surface to be lined shall be rolled with a smooth steel drum or pneumatic roller so as to be free of irregularities, loose earth and abrupt changes

in grade. In order to allow for compaction of the subgrade and liner, the subgrade surface shall be at least 12 to 24 inches above the water table surface at all times during construction. Perimeter anchor trenches shall be excavated to the lines and width shown on the plans prior to liner placement. No standing water or excessive moisture shall be allowed on the area to be lined.

C. Clay Fill:

1. The Contractor shall be responsible for locating, purchasing, transporting and stockpiling clay fill to the Astatula Ash Residue Monofill Site in accordance with the liner QA/QC plan.

2. Testing:

   a. The clay fill shall meet or exceed the specification in Paragraph 2.01 A. The Contractor shall provide soil test results of four representative samples of the fill from the borrow site. A plan showing the location of test samples taken shall be submitted to and approved in writing by the Engineer prior to sampling. Once testing is complete, the Contractor shall provide notification to the Engineer that all samples meet or exceed the requirements of Paragraph 2.01 A.

   b. If any of the test samples should fail, then the Engineer may request additional soil testing to locate acceptable zones of clay or the Engineer may request that another borrow site be located. Testing at alternate borrow sites shall follow those at the original site tested.

   c. As an alternative to the testing procedure discussed previously, the Contractor can provide a signed and sealed soil report prepared by a Professional Engineer certified in the State of Florida documenting that the fill material within the borrow site is uniform and meets or exceeds the requirements of Paragraph 2.01 A.

3. Should the Contractor stockpile the clay material at the site, he shall slope this stockpile and compact it with a track or light roller type vehicle so that the clay material will not become soft and overly saturated during rain events. Should rainfall result in overwetting of the product, it will be the Contractor's responsibility to spread, dry, and rehomogenize the product as needed prior to placement.

4. Prior to clay placement, the subgrade shall be completed to the lines and grades as shown on the Drawings and as indicated in the QA/QC plan, within ±0.10 feet or as directed by the Engineer, and shall be sufficiently wetted so that it does not absorb moisture from the overlying clay.

5. The clay shall be ready for spreading and kneading, as delivered by the supplier, but may require wetting prior to compaction. The

Contractor shall utilize a compactor-type bulldozer, such as a Caterpillar 815 which provides steel kneading feet. At the time of receipt of the clay material, the Contractor shall evenly spread, wet, if needed, and knead the product to a thickness greater than the final desired compact thickness.

6. Placement:

   a. The clay liner shall be placed in 3 lifts. Each lift shall be placed in a 6-inch layer, compacted and graded to a thickness of 3 to 5 inches. The total clay liner thickness shall be a minimum of 12 inches after total compaction. The compaction process shall provide thorough kneading throughout the entire lift. The Contractor shall measure the lines and grades of each lift and the completed clay liner and will maintain the elevation within +0.10 feet as shown on the Drawings or as directed by the Engineer.

   b. Adjacent clay panels shall be scarified at the end and overlapped to achieve a good bond.

   c. At the time of compaction, the molding moisture content in the clay shall be between the SPOMC (ASTM D-698), minus 2 percent, and the optimum moisture content, plus 5 percent. The clay lift shall be compacted to no less than 97 percent of the Modified Proctor dry density corresponding to the molding moisture content.

7. Each lift shall be compacted and tested to satisfy moisture and density requirements before a subsequent lift is placed. The surface of each completed lift (except for the final lift) shall be scarified to a depth of 2 inches prior to placing the next lift. If desiccation or crusting of the completed lift surface occurs before placement of the next lift or the HDPE liner, the area shall be scarified to the depth necessary to expose sufficiently moist materials and the scarified materials shall be brought to the correct moisture content, remixed and homogenized prior to compaction.

8. During compaction, the moisture content of the clay shall be maintained uniformly throughout the lift. Whenever the moisture content of the clay at the borrow source is lower than that specified for compaction, water shall be added and distributed homogenously within the clay. Clay material that contains excessive moisture shall be aerated by blading, discing, harrowing, or other methods, to hasten the drying process, and rehomogenized prior to compaction.

9. Damage to any compacted lift at any time during the course of construction, shall be fully repaired prior to placement of any overlying material.

10. Intermediate lifts shall be rolled to seal them when subsequent lifts will not be placed until after two calendar days. The sealed surface shall be scarified and moistened, as necessary, to prepare it for the subsequent lift.

11. Complete bonding of one clay layer to another shall take place along important contact surfaces as determined by the Engineer. At a minimum, such interfaces shall be scarified and brought to the proper moisture content prior to placement of the subsequent lift. If differences in the clay type are large, or if the contact surface is parallel to the hydraulic gradient or potential leak, the contact surface shall be stepped or keyed in order to prevent potential leakage along the contact surface.

12. Lined surfaces shall be graded and rolled so as to provide a surface free of irregularities, loose earth, and abrupt changes in grade. Construction traffic shall not be routed over the completed surface unless approved by the Engineer.

13. The surface of the final lift shall be smooth, free from roller marks, holes, depressions more than 1/2 inch deep, or protrusions extending above the surface more than 1/4 inch. The clay liner shall be free of all rocks, stones, sticks, roots, sharp objects, debris and any other deleterious materials. Stones larger than 1/4 inch in diameter, sharp-edged stones of any size and other hard objects, shall not be permitted within 6 inches of the liner surface.

14. The minimum thickness of clay lining shall be as shown on the Drawings. After placement of each lift, the clay surface shall not be allowed to dry completely. In order to prevent the formation of dessication cracks, periodic wetting may be required until the clay liner is adequately protected from the elements. Should the liner dry out, causing dessication cracks, it shall be the Contractor's responsibility to re-wet, rehomogenize, knead and recompact the product to the depth of the deepest crack to meet the intent of these specifications. A temporary protective cover of Visqueen or equal may be used over areas on which the clay layer is exposed more than 24 hours.

D. Moisture and Density Control During Compaction:

1. The measured in-place moisture content immediately after clay compaction shall be approximately equal to the SPOMC, i.e., shall be between SPOMC -2 percent and +5 percent as determined from the most recent representative Standard Proctor curve for the clay being used (ASTM D-698).

2. The measured in-place dry density immediately after clay compaction shall be equal to or greater than that stipulated in Paragraph 2.01 A from the most recent representative Modified Proctor curve for the clay being used.

3. Hydraulic Conductivity Limits: <u>All</u> hydraulic conductivities measured on undisturbed samples of the completed work shall be less than, or equal to, 1.0×10^{-8} cm/sec. If the coefficient of permeability is greater than 1×10^{-8} cm/sec., the area shall be reworked as needed though additional wetting, kneading, compaction or any

combination thereof, including product replacement. If the clay material is required to be replaced, then all areas which have been determined to have failed these limits shall be removed horizontally to the nearest passing locations and vertically to the top elevation of the nearest passing lift and shall be replaced and recompacted in properly placed lifts in compliance with all applicable conditions.

4. A temporary flexible membrane such as a thick Visqueen may be used as temporary protection for the completed natural clay liner. The temporary membrane shall be overlapped 1 foot and anchored (weighted) and does not require seaming. The temporary membrane must be removed prior to placement of the 60 mil HDPE liner.

5. Special care shall be taken to maintain the prepared clay surface and prevent it from becoming softened due to precipitation or desiccated and cracked due to lack of moisture and from flooding and freezing. No HDPE liner shall be placed in an area that has become softened by precipitation or desiccated and cracked due to lack of moisture. The clay surface shall be observed daily by the Engineer and HDPE liner installer to check for deficiencies. Any deficiencies shall be repaired by the Contractor at his expense.

3.03 TOLERANCES

A. The natural clay cover shall have a tolerance of 0.5 foot horizontal and 0.1 foot vertical, except where shown or specified as "minimum".

3.04 MOISTURE CONTROL

A. If too dry, water shall be added to material by sprinkling the fill, then homogenously mixing and kneading to achieve a uniform moisture content throughout the lift.

B. If too wet, material shall be aerated by blading, discing, harrowing, or other methods, to hasten the drying process, and then re-homogenized prior to compaction.

3.05 QUALITY CONTROL

A. Soil Testing:

1. The Contractor shall provide for the services of an independent soil testing company approved by the Engineer to perform laboratory quality control testing for clay lining materials from the borrow area.

2. Owner shall provide for the services of an independent soils testing company to perform field and laboratory quality control testing for in-place compacted clay liner materials.

3. Laboratory soil tests shall be provided in accordance with the schedule and test methods described in Paragraph 2.01 and 2.02.

4. In-place density and moisture content shall be determined at a minimum of two tests per acre per lift.

5. In addition to the soil tests, the independent soil testing company shall obtain at least one undisturbed (Shelby tube or drive-cylinder) sample of field-compacted clay lining material at a rate of one sample per acre lift of compacted imported clay placed. See Table 02212-A for frequency of tests.

6. Undisturbed samples of field compacted clay lining materials shall be permeability tested in accordance with Paragraph 2.02.

B. Topographic Surveys: Detailed topographic surveys of the site shall be performed to document suitable thickness of the installed clay lift as follows:

1. Measure elevations on a minimum 50-foot grid over the entire area and at all breaks in grade.

2. Accuracy: Within 0.1 foot vertical and 0.5 feet horizontal in accordance with national surveying standards.

3. As a minimum, surveys shall be conducted at completion of the following tasks:

   a. Subgrade preparation
   b. Clay lift placement

4. Results of topographic surveys shall be plotted in plan to a scale of 1 inch equals 50 feet and shall be submitted to the Engineer.

C. Destructive Testing:

1. Total thickness measurements to determine the thickness of the in-place finished clay layer shall be conducted every 11,000 square feet of installed liner, including tests on bottom and side slopes. Samples shall be measured by inserting a Shelby tube or drive cylinder into the clay liner. If the clay liner thickness is less than 12 inches, then the Contractor shall place additional clay to the desired 12-inch thickness, following placement procedures previously detailed in this Specification. The clay liner will then be resampled at a frequency of one test per 22,000 square feet. If any samples show a liner depth less than 12 inches, then the procedure as detailed above shall be repeated.

2. Holes in the clay liner as the result of destructive testing, shall be filled by the Contractor with the same clay material. The replacement clay fill shall be kept moist, placed in 2-inch lifts and vigorously rod-tamped into place between lifts.

PROPERTIES OF LOW PERMEABILITY CLAY LAYER

Description	Specified Value	Method	Frequency[a]
Material			
Minimum percent fines passing No. 200 sieve	25	ASTM D-1140 or D-421/ D-422	1 per 22,000 sq. ft. per lift
Atterberg limits (Liquid limit and Plasticity Index), percent	20 min. 70 max.	ASTM D-4318	1 per 44,000 sq. ft. per lift
Organic Content	--	ASTM D-2974	Initially and whenever organics are visually evident
Compaction			
Standard Proctor Compaction	-	ASTM D-698	3 at beginning of clay placement and whenever there is an apparent change in borrow material
One-point field modified Proctor	-	ASTM D-1557	1 per 22,000 sq. ft. per lift
Field moisture content relative to SPOMC	SPOMC -2% to SPOMC +5%	ASTM D-2216[b]	1 per 22,000 sq. ft. per lift
Minimum field density, as percent of Modified Proctor (ASTM D-1557)[c]	97%	ASTM D-1556[b] or ASTM D-2937	1 per 22,000 sq. ft. per lift
Maximum permeability, cm/sec	1.0×10^{-8}	d	1 per 44,000 sq. ft. per lift
Compacted thickness, initial lift Total Clay Thickness (inches)	3.0 to 5.0 12.0 min.	Direct Destructive Measurement	1 per 11,000 sq. ft. per lift

[a] Testing frequencies as listed shall be doubled for the first clay lift.

[b] Nuclear and other speedy determinations may be used when a product specific correlation has been established at the job site.

[c] Modified Proctor dry density corresponding to the molding water content.

[d] Undisturbed samples shall be encapsulated in a flexible latex membrane and tested in a triaxial type permeameter by an approved independent soil testing laboratory with back pressure to achieve saturation as specified in this section.

D. Maintenance:

1. Protect newly graded areas from traffic and erosion and keep free from trash and weeds. Repair and re-establish grades in settled, eroded, and rutted areas to the specified tolerances.

2. Where completed compacted areas are disturbed by subsequent construction operations or adverse weather, scarify the surface, reshape, re-wet as needed, rehomogenize and compact to the required density prior to further construction.

E. Settlement:

1. The Contractor shall be responsible for all settlement to backfills, fills, and embankments which may occur within the correction period stipulated in the General Conditions.

2. The Contractor shall make, or cause to be made, all repairs or replacements made necessary by settlement within 30 days after notice from the Owner or Owner's Representative.

F. Disposal of Debris: The Contractor may dispose of all debris at the landfill at no cost to the Contractor in a place designated by the Owner. However, all material must be weighed in at the landfill weigh station before disposal.

3.06 CERTIFICATION OF COMPLETION

A. Upon completion of the clay liner placement, the Contractor shall certify the following to the Owner:

1. The clay liner was constructed in accordance with the approved project plans and specifications.

2. The clay liner material meets all requirements of the approved project plans and specifications.

3. The clay liner has not been damaged during covering operation or construction.

Field and Laboratory Results

OVEN DRY MOISTURE CONTENT TESTS

Location	Moisture Content (%)
EAST LINED SPRAY EVAPORATION BASIN	
Subgrade Soil	
1. N.E. Corner	8.0
2. S.W. Corner	9.0
3. Center of W. Side Slope	9.3
Clay Liner - 1st Lift	
4. 85' S. & 16' E. of N.W. Corner	23.4
5. 76' S. & 34' E. of N.W. Corner	26.7
6. 61' N. & 32' E. of S.W. Corner	21.3
7. 64' N. & 17' W. of S.E. Corner	22.4
WEST LINED SPRAY EVAPORATION BASIN	
Clay Liner - 1st Lift	
8. 55' E. & 62' N. of S.W. Corner	21.0
9. 26' W. & 50' S. of N.E. Corner	17.8
10. 36' E. & 89' S. of N.W. Corner	19.6

ASTATULA LANDFILL
LAKE COUNTY, FLORIDA

JAMMAL & ASSOCIATES, INC. Consulting Engineers

DRAWN	SCALE	PROJ NO
PH		761-00273
CHKD	DATE	
DJD	6/90	9

CLAY LINER ATTERBERG LIMITS TEST RESULTS

Location	Liquid Limit %	Plastic Limit %	Plasticity %
Ash Basin at Lines K-8 2nd lift	47.0	16.1	30.9
Ash Basin at Lines E.5-9 2nd lift	56.0	17.8	38.2
Ash Basin at Lines M-10 2nd lift	61.9	18.2	43.7
Ash Basin at Lines 0.25-10 2nd lift	54.2	16.7	38.0
Ash Basin at Lines E-5 3rd lift	49.5	16.2	33.3
Ash Basin at Lines E-7 3rd lift	55.6	17.0	38.6
Ash Basin at Lines M-10 3rd lift	46.5	16.5	30.0
Ash Basin at Lines H-11 3rd lift	51.0	19.1	31.9
Ash Basin at Lines 0.25-10 3rd lift	58.5	19.0	39.5

Notes: Requirements: Plasticity Index - minimum 20%, maximum 70%

ASTATULA LANDFILL CLAY LINER
LAKE COUNTY, FLORIDA

JAMMAL & ASSOCIATES, INC. Consulting Engineers

DRAWN	SCALE	PROJ NO 761-00273
DJD	DATE 8/90	1

CLAY LINER ATTERBERG LIMITS TEST RESULTS

Location	Liquid Limit %	Plastic Limit %	Plasticity %
East lined spray evaporation basin, 1st lift 61' N. & 32' E. of S.W. Corner	62.5	22.6	39.9
East lined spray evaporation basin, 1st lift 64' N. & 17' W. of S.E. Corner	68.5	23.3	45.2
East lined spray evaporation basin, 2nd lift 43' S. & 50' E. of N.W. Corner	43.6	17.2	26.4
East lined spray evaporation basin, 3rd lift 62' S. & 53' E. of N.W. Corner	60.6	19.2	41.4
West lined spray evaporation basin, 1st lift 26' W. & 50' S. of N.E. Corner	44.2	18.6	25.6
West lined spray evaporation basin, 1st lift 55' E. & 62' N. of S.W. Corner	56.6	20.5	36.1
West lined spray evaporation basin, 3rd lift 37' S. & 43' E. of N.W. Corner	54.6	19.4	35.2
West lined spray evaporation basin, 3rd lift 39' N. & 47' E. of S.W. Corner	55.2	18.7	36.5

Notes: Requirements: Plasticity Index - minimum 20%, maximum 70%

ASTATULA LANDFILL CLAY LINER
LAKE COUNTY, FLORIDA

JAMMAL & ASSOCIATES, INC. Consulting Engineers

DRAWN	SCALE	PROJ NO 761-00273
CHKD DJD	DATE 7/90	2

CLAY LINER ATTERBERG LIMITS TEST RESULTS

Location	Liquid Limit %	Plastic Limit %	Plasticity %
Ash Basin at Lines G-8 1st lift	54.0	17.1	36.9
Ash Basin at Lines E-7 1st lift	54.7	19.0	35.7
Ash Basin at Lines H-3.5 1st lift	46.7	16.3	30.4
Ash Basin at Lines I-5 1st lift	56.7	18.9	37.8
Ash Basin at Lines K.5-2.5 2nd lift	46.4	17.3	29.1
Ash Basin at Lines K-8 3rd lift	49.7	19.4	30.3
Proctor Sample (78.1 pcf @ 22.9% M.C.)	59.2	24.4	34.8

Notes: Requirements: Plasticity Index - minimum 20%, maximum 70%

ASTATULA LANDFILL CLAY LINER
LAKE COUNTY, FLORIDA

JAMMAL & ASSOCIATES, INC. Consulting Engineers

DRAWN	SCALE	PROJ NO 761-00273
CHKD DJD	DATE 7/90	3

ASH RESIDUE MONOFILL BASIN

CLAY LINER

-200 SIEVE TEST RESULTS

Location	% Passing -200 Sieve
Ash Residue Monofill Basin	
1st Lift	
At Lines G-8	46.2
At Lines I-5	48.7
At Lines H-3.5	38.3
At Lines E-7	48.9
At Lines H-3.5	37.7
2nd Lift	
At Lines M-10	54.2
At Lines E.5-9	45.4
At Lines K-8	41.8
At Lines K.5-2.5	39.6
3rd Lift	
At Lines K-8	45.1
At Lines E-7	45.1
At Lines E-5	41.8
At Lines M-10	47.0
At Lines H-11	47.6
East Lined Spray Evaporation Basin	
3rd Lift	
N.W. Corner	49.7

Project Requirements: Minimum 25%

ASTATULA LANDFILL

LAKE COUNTY, FLORIDA

JAMMAL & ASSOCIATES, INC. Consulting Engineers

DRAWN PH	SCALE	PROJ NO	761-00273
CHKD DJD	DATE 6/90		Sheet 7

-200 SIEVE TEST RESULTS

Location	% Passing -200 Sieve
EAST LINED SPRAY EVAPORATION BASIN	
Clay Liner - 1st Lift	
85' S. & 16' E. of N.W. Corner	46.0
44'N & 37' E. of S.W. Corner	40.0
53' S. & 29' E. of N.W. Corner	35.1
Clay Liner - 2nd Lift	
41' S. & 19' E. of N.W. Corner	41.6
50' N. & 54' E. of S.W. Corner	42.7
43' S. & 50' E. of N.W. Corner	40.3
WEST LINED SPRAY EVAPORATION BASIN	
Clay Liner - 1st Lift	
55' E. & 62' N. of S.W. Corner	49.0
26' W. & 50' S. of N.E. Corner	33.2
36' E. & 89' S. of N.W. Corner	42.4
Clay Liner - 2nd Lift	
47' W. & 43' S. of N.E. Corner	46.3
67' N. & 24' W. of S.E. Corner	48.9
Clay Liner - 3rd Lift	
39' N. & 47' E. of S.W. Corner	47.7
37' S. & 43' E. of N.W. Corner	46.6

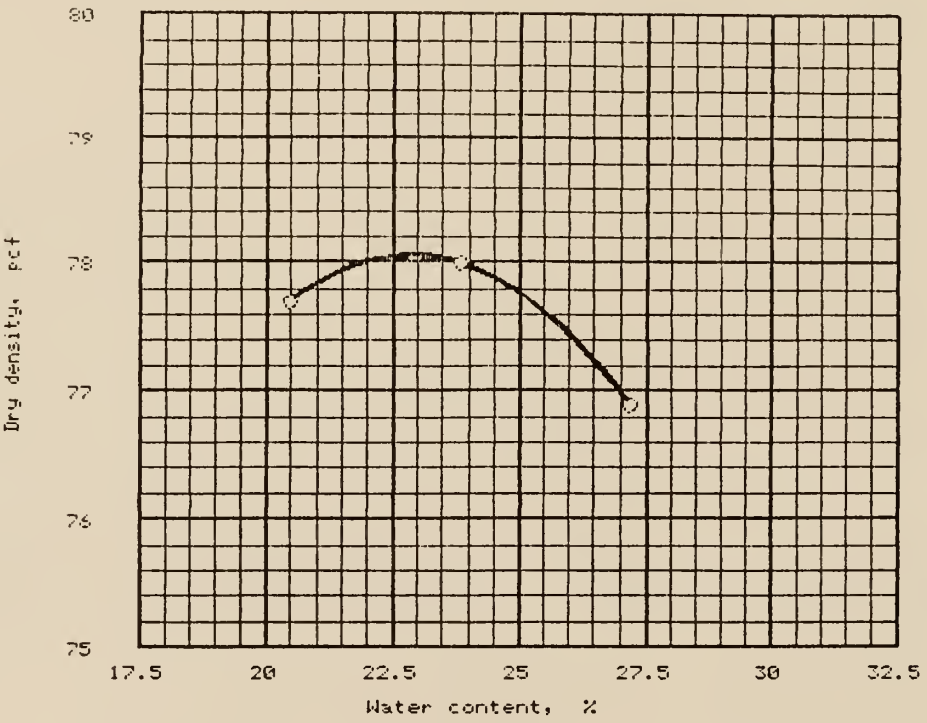
Note: 1) Minimum requirement: 25% passing No. 200 Sieve

ASTATULA LANDFILL
LAKE COUNTY, FLORIDA

JAMMAL & ASSOCIATES, INC. Consulting Engineers

DRAWN	PH	SCALE	PROJ NO 761-00273
CHKD	DJD	DATE 6/90	8

PROCTOR TEST REPORT



"Standard" Proctor, ASTM D 698, Method A

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	SP-SC							

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 22.9 % Maximum dry density = 78.1 pcf	Lt. Brown to Lt. Yellow-Brown Clayey F/S

Project No.: 761-00273
Project: Astatula Landfill
Location:

Date: 5-15-1990

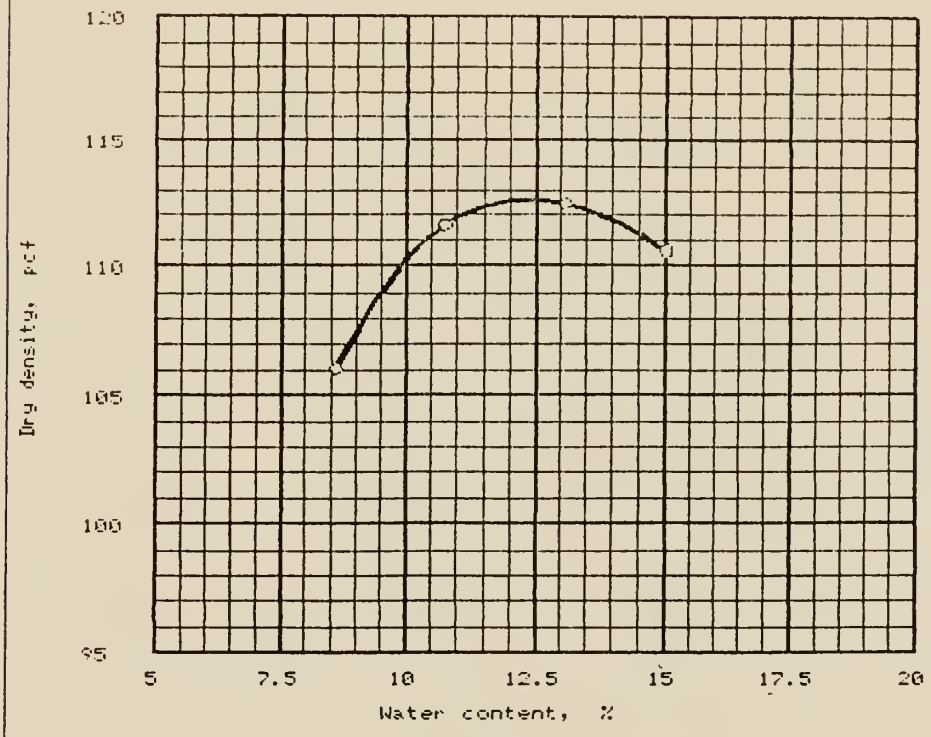
Remarks:
DJD

PROCTOR TEST REPORT

JAMMAL & ASSOCIATES, INC.

Figure No. 1

PROCTOR TEST REPORT



"Standard" Proctor, ASTM D 698, Method A

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	SP-SC							

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 12.4 % Maximum dry density = 112.6 pcf	Orange Clayey F/S

Project No.: 761-00273
Project: Astatula Landfill
Location: West Lined Spray Evaporation Basin

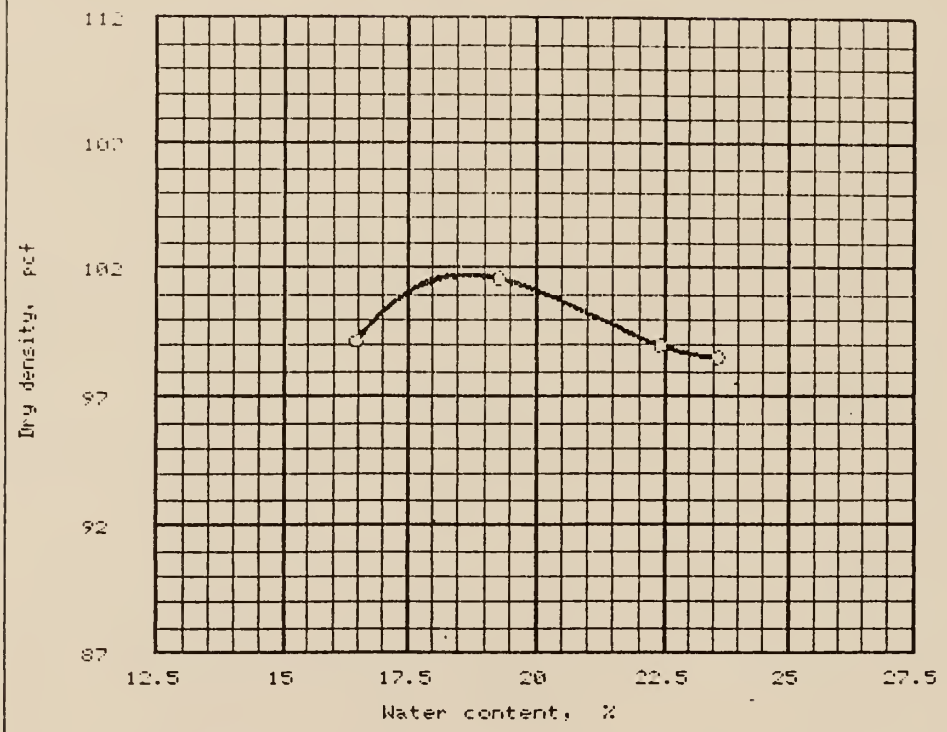
Date: 5-15-1990

Remarks:
DJD

PROCTOR TEST REPORT
JAMMAL & ASSOCIATES, INC.

Figure No. 2

PROCTOR TEST REPORT



"Modified" Proctor, ASTM D 1557, Method A

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 18.7 % Maximum dry density = 101.7 pcf	Lt. brn. to yellowish brn. silty clay

Project No.: 761-00273
Project: Astatula Landfill
Location: East lined spray evaporation basin

Date: 5-18-1990

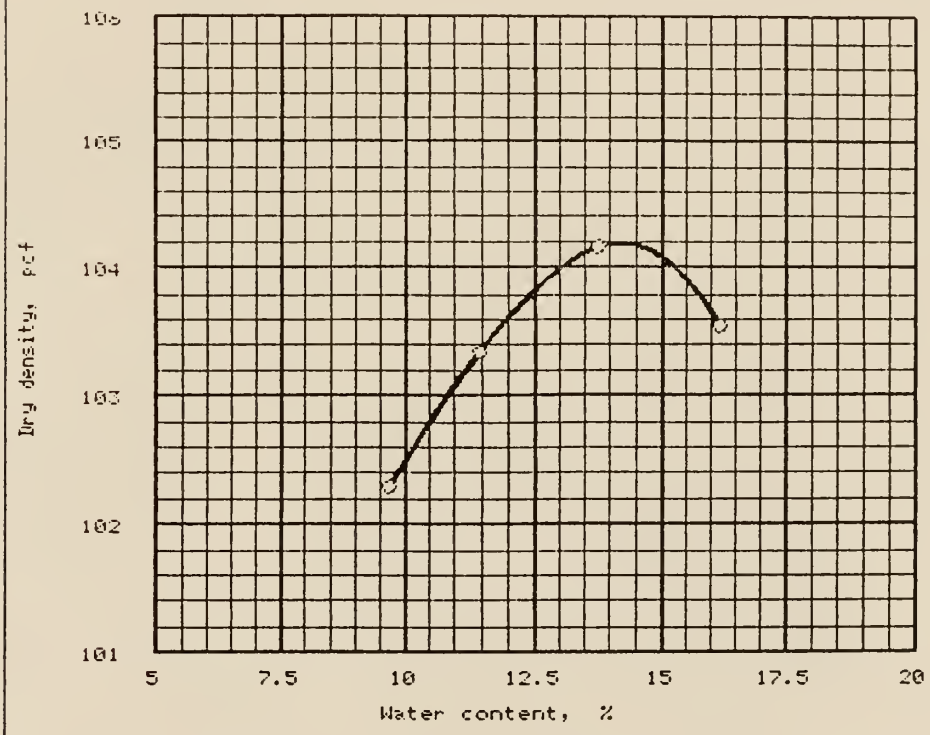
PROCTOR TEST REPORT

JAMMAL & ASSOCIATES, INC.

Remarks:

Figure No. 3

PROCTOR TEST REPORT

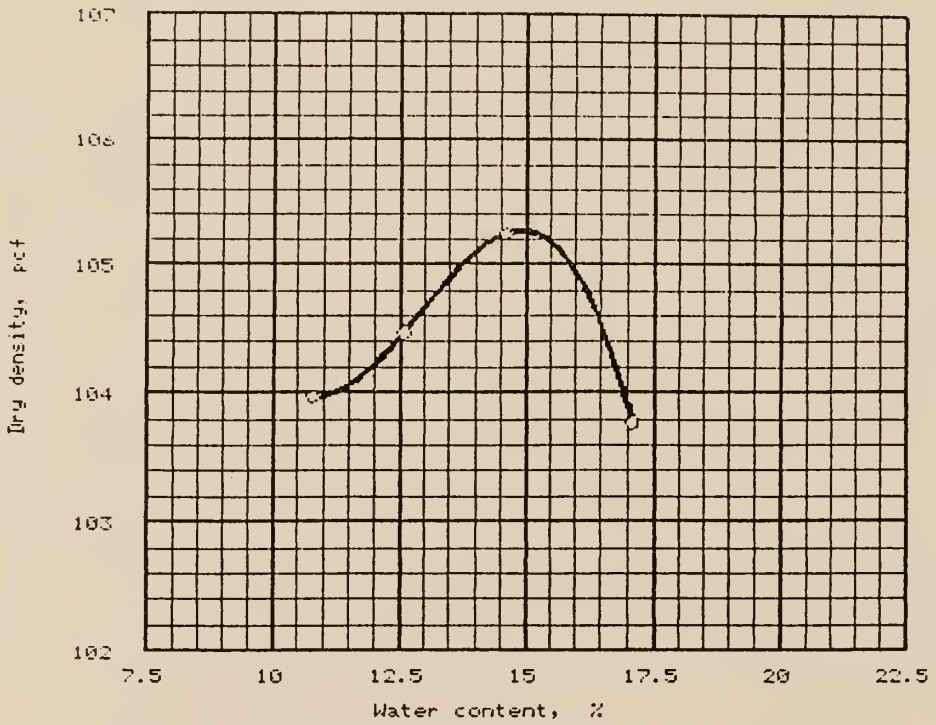


"Modified" Proctor, AASHTO T180, Method A

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	SP							

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 14.2 % Maximum dry density = 104.2 pcf	Lt. Brown to Yellowish Brown F/S
Project No.: 761-00273 Project: Astatula Landfill Location: Between S-11 & S-12 Date: 5-01-1990	Remarks: DJD
PROCTOR TEST REPORT JAMMAL & ASSOCIATES, INC.	Figure No. 4

PROCTOR TEST REPORT

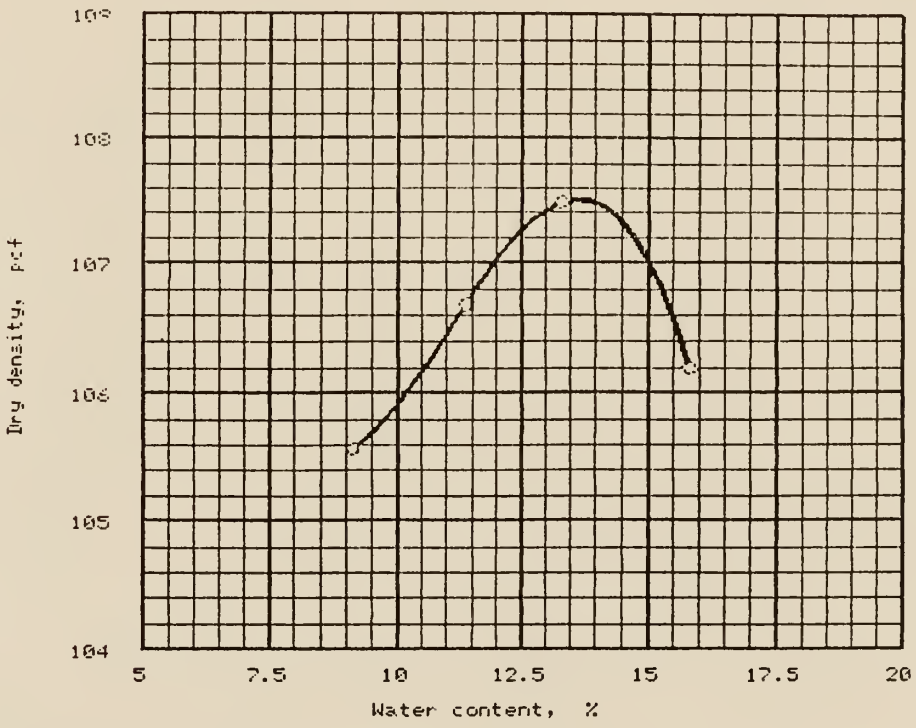


"Modified" Proctor, AASHTO T180, Method A

Elev./ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	SP							

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 14.9 % Maximum dry density = 105.3 pcf	Brown to Light Brown Fine Sand
Project No.: 761-00273 Project: Astatula Landfill Location: Ash Residue Monofill Basin North Slope Date: 5-22-1990	Remarks: DJD
PROCTOR TEST REPORT JAMMAL & ASSOCIATES, INC.	Figure No. 5

PROCTOR TEST REPORT



"Modified" Proctor, AASHTO T180, Method A

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	SP-SM							

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 13.7 % Maximum dry density = 107.5 pcf	Brown to Lt. Brown F/S with Orange S/S F/S

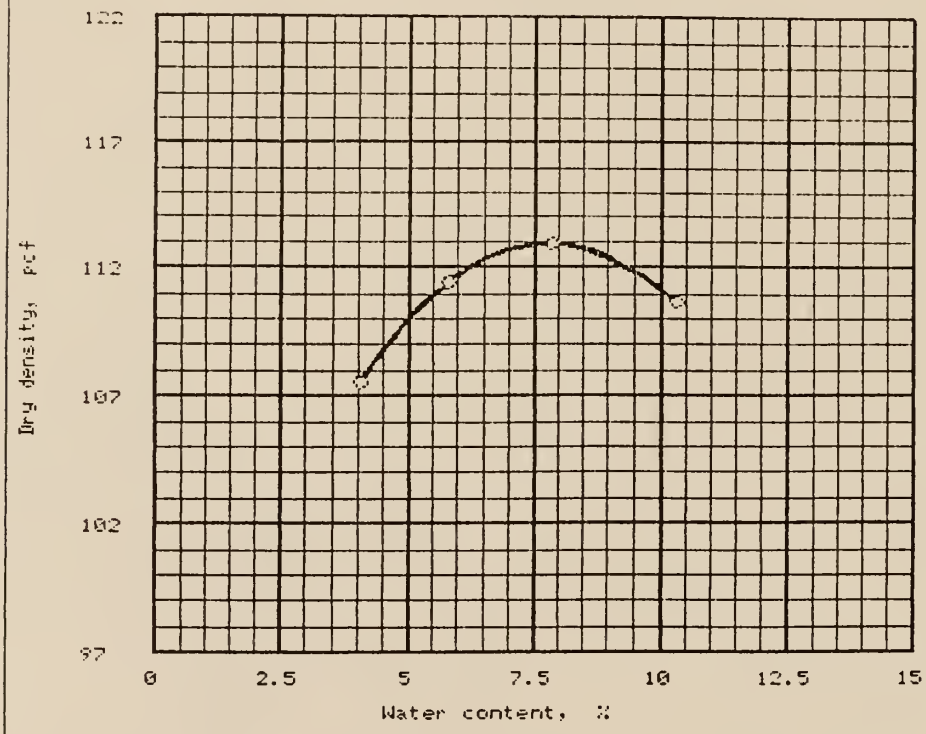
Project No.: 761-00273
Project: Astatula Landfill
Location: Ash Residue Monofill Basin
North Slope
Date: 5-22-1990

Remarks:
DJD

PROCTOR TEST REPORT
JAMMAL & ASSOCIATES, INC.

Figure No. 6

PROCTOR TEST REPORT



"Modified" Proctor, ASTM D 1557, Method A

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 7.8 % Maximum dry density = 112.9 pcf	Orange slightly silty fine sand

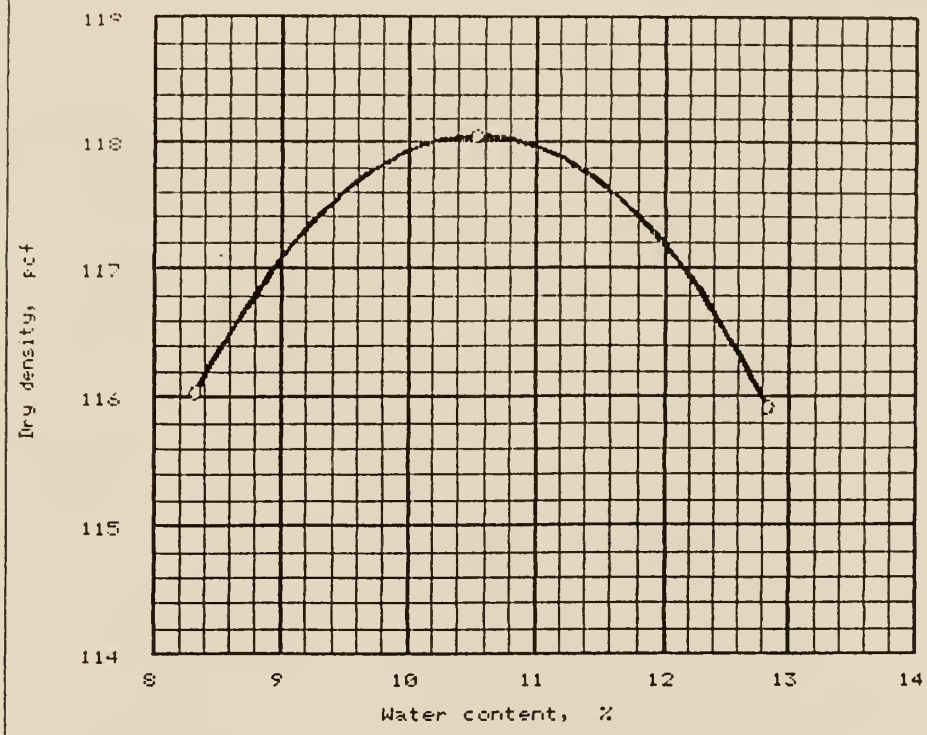
Project No.: 781-00273
Project: Astatula Landfill
Location: Ash residue monofill basin
slope- grid #1
Date: 5-21-1990

Remarks:
DD

PROCTOR TEST REPORT
JAMMAL & ASSOCIATES, INC.

Figure No. 7

PROCTOR TEST REPORT



"Modified" Proctor, ASTM D 1557, Method A

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	SM							

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 10.5 % Maximum dry density = 118.1 pcf	ORANGE CAYEY FINE SAND

Project No.: 761-00-273
Project: ASTATULA LANDFILL
Location: ASH RESIDUE BASIN ACCESS ROAD WEST SIDE
Date: 5-14-1990

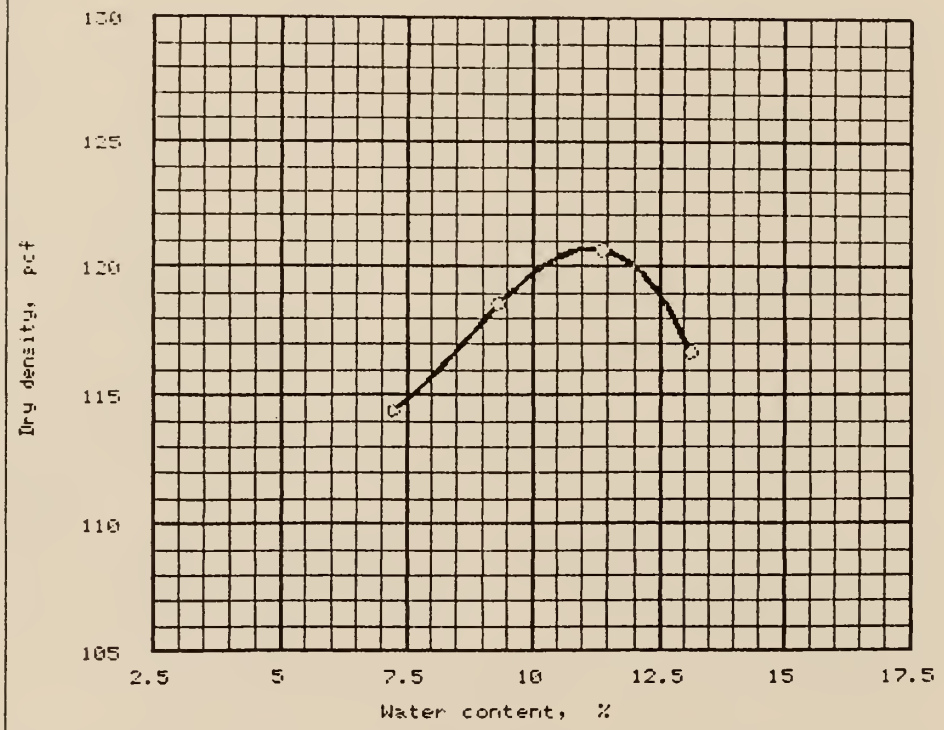
PROCTOR TEST REPORT

JAMMAL & ASSOCIATES, INC.

Remarks:

Figure No. 8

PROCTOR TEST REPORT



"Modified" Proctor, ASTM D 1557, Method A

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
	SM							

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 11.2 % Maximum dry density = 120.7 pcf	ORANGE CLAYEY FINE SAND

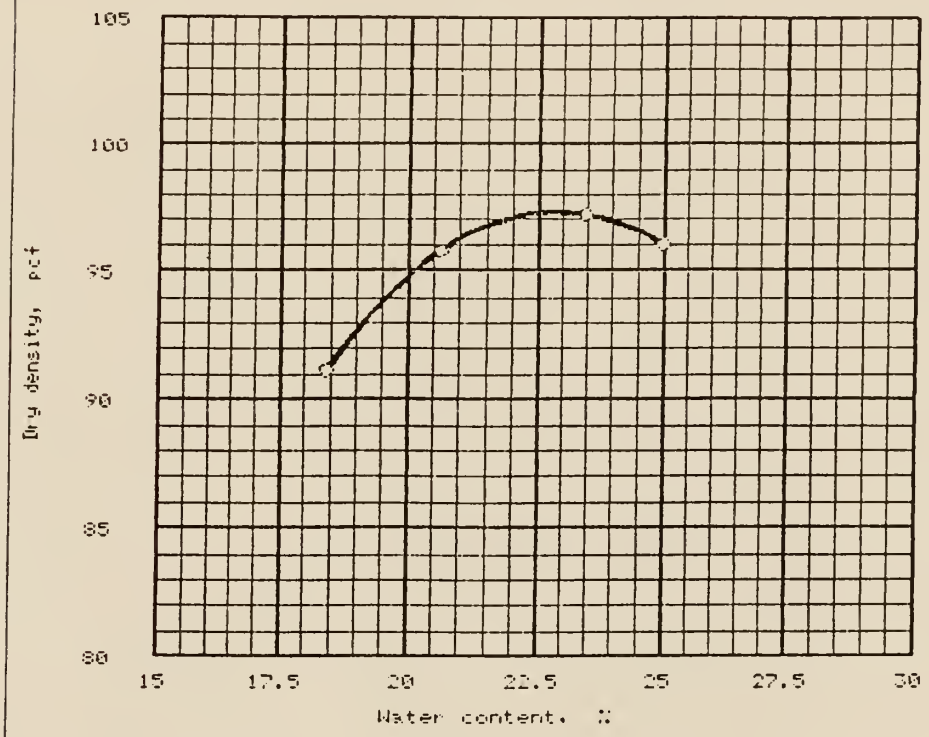
Project No.: 761-00-273
Project: ASTATULA LANDFILL
Location: ASH RESIDUE BASIN ACCESS ROAD NORTH EAST CORNER
Date: 5-14-1990

Remarks:

PROCTOR TEST REPORT
JAMMAL & ASSOCIATES, INC.

Figure No. 9

PROCTOR TEST REPORT



"Standard" Proctor, ASTM D 698, Method A

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	SP-SC							

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 22.9 % Maximum dry density = 97.3 pcf	Light Brown & Brown Silty Clayey Sand

Project No.: 781-00273
Project: Astatula Landfill
Location: Pit

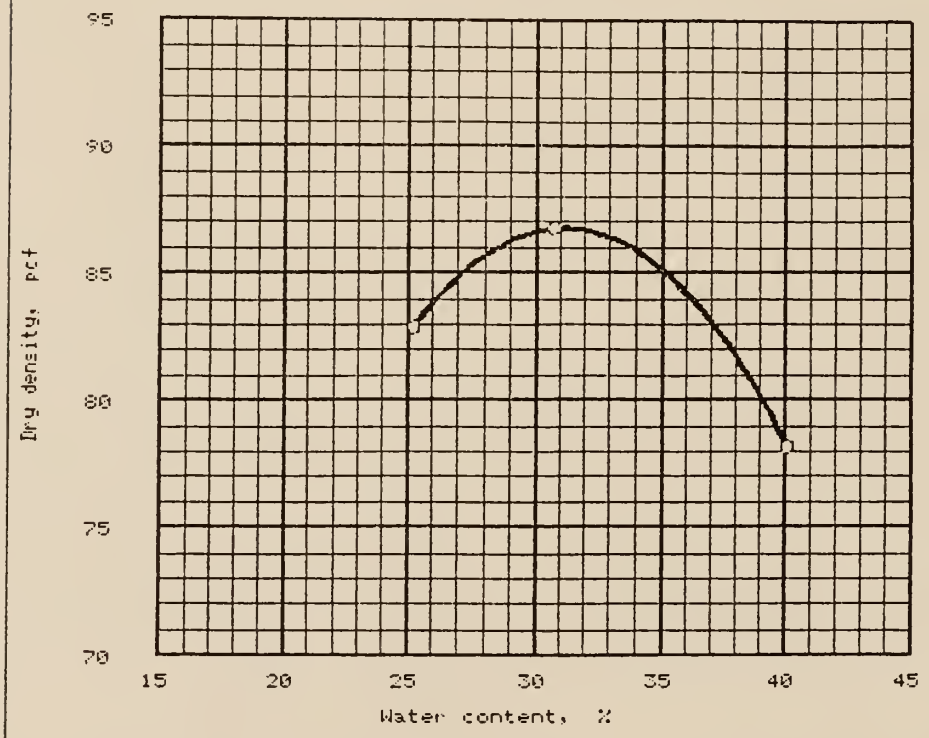
Date: 6-11-1990

Remarks:
DJD

PROCTOR TEST REPORT
JAMMAL & ASSOCIATES, INC.

Figure No. 1

PROCTOR TEST REPORT



"Standard" Proctor, ASTM D 698, Method A

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	SP-SC							

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 11.2 % Maximum dry density = 86.8 pcf	Yellow Clayey Sand

Project No.: 761-00273
Project: Astatula Landfill
Location:

Date: 8-30-1990

Remarks:
DJD

PROCTOR TEST REPORT

JAMMAL & ASSOCIATES, INC.

Figure No. 1

CLAY LINER PERMEABILITY TEST RESULTS

Location	Sample Date	Length (cm)	Area cm	Dry Unit Weight (pcf)	Moisture Content %	Hydraulic Gradient	Degree of Saturation %	Max. Cell Pressure & Back Pressure (psi)	Hydraulic Conductivity (cm/sec)
East Lined Spray Evaporation Basin - 1st lift 76' S. & 34' E. of N.W. Corner	5-15-90	6.2	41.88	94.5	26.7	34.04	100	93/90	5.6×10^{-8}
West Lined Spray Evaporation Basin - 2nd lift 67' N. & 24' W. of S.E. Corner	5-18-90	5.3	41.88	106.4	22.3	39.82	100	93/90	3.6×10^{-8}

Notes: 1) Sampling method - thin walled Shelby tube
2) Requirement - Maximum 1.0×10^{-8} cm/sec

ASTATULA LANDFILL CLAY LINER
LAKE COUNTY, FLORIDA

JAMMAL & ASSOCIATES, INC. Consulting Engineers

DRAWN	SCALE	PROJ NO
		761-00273
CHKD DJD	DATE 6/90	1

Table 1

SUMMARY OF LABORATORY TEST RESULTS

		Drive Cylinder Sample				Permeability Test Specimen						
Sample Number	Sample Location	Dry Density (lb/ft^3)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Initial Dry Density (lb/ft^3)	Initial Moisture Content (%)	Final Dry Density (lb/ft^3)	Final Moisture Content (%)	Fines Content -200 (%)	Degree of Saturation (%)	Coefficient of Permeability (cm/sec)
A	East Basin 1st lift 76'S & 34'E of NW Corner	95.1	24.2 to 32.4 (ave.=28.3)	53	33	94.1	27.8	90.2	32.7	60	100*	1.1×10^{-9}
B	West Basin 2nd lift 67'N & 24'W of SE Corner	111.0	17.5 to 19.8 (ave.=18.7)	57	39	111.0	19.6	105.6	22.2	47	100**	7.9×10^{-10}

*The calculated degree of saturation was 102 percent, assuming a specific gravity of 2.7.
**The calculated degree of saturation was 101 percent, assuming a specific gravity of 2.7.

CLAY LINER PERMEABILITY TEST RESULTS

Location	Sample Date	Length (cm)	Area cm	Dry Unit Weight (pcf)	Moisture Content %	Hydraulic Gradient	Degree of Saturation %	Max. Cell Pressure & Back Pressure (psi)	Hydraulic Conductivity (cm/sec)
East Lined Spray Evaporation Basin - 1st lift 64' N. & 17' W. of S.E. Corner	5-15-90	3.1	41.88	100.7	22.4	22.7	100	95/90	5.0×10^{-9}
West Lined Spray Evaporation Basin - 1st lift 55' E. & 62' N. of S.W. Corner	5-16-90	3.9	41.88	104.6	21.0	17.8	100	95/90	8.2×10^{-9}

Notes: 1) Sampling method - thin walled Shelby tube
2) Requirement - Maximum 1.0×10^{-8} cm/sec

ASTATULA LANDFILL CLAY LINER
LAKE COUNTY, FLORIDA

JAMMAL & ASSOCIATES, INC. Consulting Engineers

DRAWN	SCALE	PROJ NO
		761-00273
CHKD DJD	DATE 6/90	4

CLAY LINER PERMEABILITY TEST RESULTS

Location	Sample Date	Length (cm)	Area cm^2	Dry Unit Weight (pcf)	Moisture Content %	Hydraulic Gradient	Degree of Saturation %	Max. Cell Pressure & Back Pressure (psi)	Hydraulic Conductivity (cm/sec)
Ash Basin									
1st Lift									
At lines 0.25-3.5	5-30-90	4.5	41.88	104.5	24.1	109.4	100	100/90	3.9×10^{-9}
At lines I-5	5-31-90	3.8	41.88	107.8	21.9	72.5	100	93/90	3.2×10^{-9}
At lines 0.25-10	5-31-90	4.1	41.88	100.6	23.6	67.4	100	93/90	9.4×10^{-9}
2nd Lift									
At lines F-5	6-1-90	4.2	41.88	101.5	26.7	65.8	100	93/90	4.4×10^{-9}
At lines L.5-5	6-1-90	4.4	41.88	107.0	21.0	62.2	100	93/90	7.6×10^{-9}
At lines K-8	6-5-90	4.0	41.88	101.9	23.2	69.2	100	93/90	9.8×10^{-9}
At lines K.5-2.5	5-29-90	4.6	41.88	106.6	21.2	76.5	100	95/90	4.1×10^{-9}
3rd Lift									
At lines E-5	6-2-90	4.4	41.88	104.8	23.2	81.0	100	95/90	2.4×10^{-9}
At lines E-7	6-6-90	4.4	41.88	107.7	21.1	62.2	100	93/90	7.4×10^{-9}
At lines H-11	6-20-90	4.3	41.88	105.4	21.1	64.7	100	93/90	5.4×10^{-9}
At lines E.5-9	6-14-90	4.0	41.88	99.1	24.9	69.0	100	99/90	7.8×10^{-9}

Notes: 1) Sampling Method - thin walled shelby tube
2) Requirement - maximum 1.0×10^{-8} cm/sec

ASTATULA LANDFILL CLAY LINER
LAKE COUNTY, FLORIDA

JAMMAL & ASSOCIATES, INC. Consulting Engineers

DRAWN	SCALE	PROJ NO
		761-00273
CHKD	DATE	
DJD	8/90	1

CLAY LINER PERMEABILITY TEST RESULTS

Location	Sample Date	Length (cm)	Area cm^2	Dry Unit Weight (pcf)	Moisture Content %	Hydraulic Gradient	Degree of Saturation %	Max. Cell Pressure & Back Pressure (psi)	Hydraulic Conductivity (cm/sec)
Ash Basin									
1st Lift									
At lines H-11	6-2-90	4.6	41.88	106.3	21.5	152.9	100	100/90	4.1×10^{-9}
At lines G-8	6-1-90	4.5	41.88	105.5	23.1	77.8	100	95/90	3.3×10^{-9}
2nd Lift									
At lines M-10	6-15-90	4.3	41.88	98.5	25.1	81.2	100	95/90	3.3×10^{-9}

Notes: 1) Sampling Method - thin walled shelby tube
2) Requirement - maximum 1.0×10^{-8} cm/sec

ASTATULA LANDFILL CLAY LINER
LAKE COUNTY, FLORIDA

JAMMAL & ASSOCIATES, INC. Consulting Engineers

DRAWN	SCALE	PROJ NO
		761-00273
CHKD DJD	DATE 8/90	1

DRAINAGE SAND PERMEABILITY TEST RESULTS

Location	Permeability (cm/sec)
Ash Residue Monofill Basin	
At lines H-7	1.0×10^{-2}
At lines J-3.5	2.1×10^{-3}
At lines K.5-9	8.6×10^{-3}
At lines H-9	6.3×10^{-3}

Note: Permeability Requirement: minimum 1×10^{-3} cm/sec

ASTATULA LANDFILL CLAY LINER
LAKE COUNTY, FLORIDA

JAMMAL & ASSOCIATES, INC. Consulting Engineers

DRAWN	SCALE	PROJ NO 761-00273
CHKD DJD	DATE 7/90	5

Date 1990	Location of Test	OMC %	Max. Den lb/cu. ft.	Field Moisture %	Field Density lb/cu. ft.	Percent of Max density	Depth of Test
	SUBGRADE SOIL						
	Ash Residue Monofill Basin						
	Slopes						
5-24	At Lines F-9	7.8	112.9	8.8	106.7	94.5	0-1
5-24	At Lines 10.5	13.7	107.5	9.1	98.2	91.4*	0-1
5-29	" -Retest	7.8	112.9	8.4	107.6	95.3**	0-1
5-29	At Lines H-11	7.8	112.9	11.0	105.0	93.0	0-1
6-1	At Lines K.5-11	7.8	112.9	7.1	106.5	94.3	0-1

NOTES:
1. Minimum compaction requirement 92% of a Modified Proctor Value
2. Depth of test referenced in feet to top of subgrade.

*Test results fail to meet minimum requirement. **Retest results meet requirement.

ASTATULA LANDFILL
LAKE COUNTY, FLORIDA

RESULTS OF FIELD COMPACTION TESTS

JAMMAL & ASSOCIATES, INC. Consulting Engineers

Tested by: PH	Date:	Project No.	761-00273
Checked by: DJD	Date: 6/90	Sheet No.	1

Date 1990	Location of Test	OMC %	Max. Den. lb/cu. ft.	Field Moisture %	Field Density lb/cu ft.	Percent of Max. density	Depth of Test
	CLAY LINER						
	Ash Residue Monofill Basin						
	1st Lift						
5-30	At Lines E-5 (Sloped Area)	25.3	104.1	25.3	98.7	94.8*	0-5
5-30	" -Retest	25.3	104.1	25.3	103.8	99.7**	0-5
5-30	At Lines E-7 (Sloped Area)	27.1	101.6	27.1	98.5	97.5	0-5
5-30	At Lines 0.25-3.5 (Sloped Area)	24.1	105.3	24.1	99.1	94.2*	0-5
5-30	" -Retest	24.1	105.3	24.1	104.5	99.2**	0-5
5-30	At Lines 0.25-6.0 (Sloped Area)	22.1	106.6	22.1	102.3	95.9*	0-5
5-30	" -Retest	22.1	106.6	22.1	103.2	96.8**	0-5
5-31	At Lines F-5	21.0	106.9	21.0	107.6	100+	0-5
5-31	At Lines I-5	21.9	107.3	21.9	107.8	100+	0-5
5-31	At Lines E-6	26.1	101.1	26.1	103.8	100+	0-5
5-31	At Lines K-6	20.7	105.6	20.7	108.6	100+	0-5
5-31	At Lines 0.25-8 (Sloped Area)	22.1	107.6	22.1	104.1	96.8	0-5
5-31	At Lines 0.25-10 (Sloped Area)	23.6	103.8	23.6	100.6	96.9	0-5
6-1	At Lines G-8	23.1	107.9	23.1	105.5	97.7	0-4
6-1	At Lines K-8	22.5	111.2	22.5	109.2	98.2	0-4

NOTES:
1. Minimum compaction requirement 97% of a 1 Point Modified Field Proctor
2. Depth of test referenced in inches to top of clay lift.

*Test results fail to meet minimum requirement. **Retest results meet requirement.

ASTATULA LANDFILL
LAKE COUNTY, FLORIDA

RESULTS OF FIELD COMPACTION TESTS

JAMMAL & ASSOCIATES, INC. Consulting Engineers

Tested by: PH	Date:	Project No.	761-00273
Checked by: DJD	Date: 6/90	Sheet No.	2

Date 1990	Location of Test	OMC %	Max. Den. lb/cu. ft.	Field Moisture %	Field Density lb/cu. ft.	Percent of Max. density	Depth of Test
	CLAY LINER						
	Ash Residue Monofill Basin						
	1st Lift						
6-2	At Lines H-11 (Sloped Area)	21.5	108.6	21.5	106.3	97.8	0-4
6-2	At Lines 0.25-10 (Sloped Area)	23.1	106.7	23.1	103.8	97.3	0-4
6-2	At Lines 0.25-8 (Sloped Area)	21.6	107.5	21.6	107.2	99.7	0-4
6-2	At Lines K.5-11 (Sloped Area)	22.4	106.6	22.4	105.4	98.8	0-4
6-5	At Lines I-9	23.3	107.0	23.3	104.5	97.7	0-4
6-5	At Lines N-9	24.7	101.2	24.7	101.4	100+	0-4
6-6	At Lines I-10	25.6	102.7	25.6	101.6	98.9	0-4
6-6	At Lines M-10	20.9	108.1	20.9	105.8	97.8	0-4
6-14	At Lines E.5-9	23.2	105.4	23.2	103.6	98.2	0-4
6-15	At LInes F-10.5	21.3	104.1	21.3	101.0	97.0	0-4

NOTES:
1. Minimum compaction requirement 97% of a 1 Point Modified Field Proctor
2. Depth of test referenced in inches to top of clay lift.

*Test results fail to meet minimum requirement. **Retest results meet requirement.

ASTATULA LANDFILL
LAKE COUNTY, FLORIDA

RESULTS OF FIELD COMPACTION TESTS

JAMMAL & ASSOCIATES, INC. Consulting Engineers

Tested by: PH		Date:		Project No.	761-00273
Checked by: DJD		Date:	6/90	Sheet No.	3

Date	Location of Test	OMC %	Max. Den. lb/cu. ft.	Field Moisture %	Field Density lb/cu. ft.	Percent of Max. density	Depth of Test
1990							
	CLAY LINER						
	Ash Residue Monofill Basin						
	3rd Lift						
5-30	At Lines F-2.5 (Sloped Area)	21.7	106.5	21.7	107.3	100+	0-4
5-30	At Lines K.5-2.5 (Sloped Area)	21.8	104.8	21.8	104.6	99.8	0-4
6-1	At Lines 0.25-6 (Sloped Area)			19.8*			
6-1	" -Retest			19.5*			
6-2	At Lines E-5 (Sloped Area)	23.2	105.4	23.2	104.8	99.4	0-4
6-2	At Lines F-5	20.8	109.5	20.8	108.2	98.8	-4
6-2	At Lines E-7			20.1*			
6-6	" -Retest	21.1	108.0	21.1**	107.7	99.7	0-4
6-5	At Lines L.5-5	23.5	103.7	23.5	100.6	97.0	0-4
6-5	At Lines I-5	21.8	108.6	21.8	106.8	98.3	0-4
6-7	At LInes G-8	24.0	103.8	24.0	103.5	99.7	0-4
6-7	At Lines K-8	24.5	100.7	24.5	100.7	100.0	0-4
6-13	AT Lines 0.25-3.5	21.2	109.1	21.2	103.8	95.1*	0-4
6-13	" -Retest	21.7	106.0	21.7	103.4	97.5**	0-4
6-14	At Lines E.5-9	24.9	101.3	24.9	99.1	97.8	0-4
6-16	At Lines I-10	21.5	107.1	21.5	160.5	99.4	0-4
6-16	At Lines M-10	20.9	109.9	20.9	110.7	100+	0-4
6-20	At Lines H-11 (Sloped Area)	21.1	108.0	21.1	105.4	97.6	0-4

NOTES:
1. Minimum compaction requirement 97% of a 1 Point Modified Field Proctor
2. Depth of test referenced in inches to top of clay lift.

*Test results fail to meet minimum requirement. **Retest results meet requirement.

ASTATULA LANDFILL
LAKE COUNTY, FLORIDA

RESULTS OF FIELD COMPACTION TESTS

JAMMAL & ASSOCIATES, INC. Consulting Engineers

Tested by: PH	Date:	Project No.	761-00273
Checked by: DJD	Date: 6/90	Sheet No.	5

CLAY LINER

ASH RESIDUE MONOFILL BASIN

THICKNESS MEASUREMENTS

Date (1990)	Location	In-Place Thickness (in)
6-1	At Lines H-2.5	14+
6-1	At Lines M.5-2.5	14+
6-7	At Lines H-5	13.5
6-7	At Lines L.5-5	12.5
6-13	At Lines O-5	14+
6-13	At Lines O.25-5	13.75
6-8	At Lines E-7	14+
6-8	At Lines J-7	12.5
6-15	At Lines O.25-7	14+
6-16	At Lines E-9	14+
6-16	At Lines E.5-9	12
6-16	At Lines F-9	14+
6-20	At Lines I-9.5	14
6-20	At Lines K-9.5	12
6-20	At Lines M-9.5	11
6-20	" -Retest	11.5
6-20	At Lines M.5-9.5	10
6-20	" -Retest	11.5
6-21	At Lines H-11	14+
6-21	At Lines K.5-11	14
7-3	At Lines O.25-10	14+

Thickness Requirement: 12"

ASTATULA LANDFILL
LAKE COUNTY, FLORIDA

JAMMAL & ASSOCIATES, INC. Consulting Engineers

DRAWN PH	SCALE	PROJ NO 761-00273	
CHKD DJD	DATE 6/90	Sheet 6	

Date 1990	Location of Test	OMC %	Max. Den. lb/cu. ft.	Field Moisture %	Field Density lb/cu. ft.	Percent of Max. density	Depth of Test
	CLAY LINER						
	Ash Residue Monofill Basin						
	3rd Lift						
6-29	At lines 0.25-10 (sloped area)	21.0	110.0	21.0	103.0	93.6*	0-4
6-30	" Retest	24.0	107.4	24.0	100.0	93.2*	0-4
7-3	" Retest	23.2	106.0	23.2	103.4	97.5**	0-4

NOTES:
1. Minimum compaction requirement 97% of a Modified Proctor Value
2. Depth of test referenced in inches to top of clay lift.

*Test results below minimum requirement
**Retest results meet minimum requirement.

ASTATULA LANDFILL CLAY LINER
LAKE COUNTY, FLORIDA

RESULTS OF FIELD COMPACTION TESTS

JAMMAL & ASSOCIATES, INC. Consulting Engineers

Tested by: PH	Date:	Project No.	761-00273
Checked by: DJD	Date: 7/90	Sheet No.	1

Date 1990	Location of Test	OMC %	Max. Den. lb/cu. ft.	Field Moisture %	Field Density lb/cu. ft.	Percent of Max. density	Depth of Test
	CLAY LINER WITH SORBOND						
	Top Lift						
	West Lined Spray Evaporation Basin						
8-28	40' S. & 30' E. of N.W. Corner	33.2	88.5	33.2	88.5	100.0	0-4
	East Lined Evaporation Basin						
8-29	130' S. & 50' W. of N.E. Corner	28.2	99.7	28.2	96.9	97.2	0-4

NOTES:
1. Minimum compaction requirement 97% of a 1 Point Modified Field Proctor
2. Depth of test referenced in inches to top of liner.

ASTATULA LANDFILL CLAY LINER
LAKE COUNTY, FLORIDA

RESULTS OF FIELD COMPACTION TESTS

JAMMAL & ASSOCIATES, INC. Consulting Engineers

Tested by: PH	Date:		Project No.	761-00273
Checked by: DJD	Date:	8/90	Sheet No.	2

Date 1990	Location of Test	OMC %	Max Den lb/cu ft	Field Moisture %	Field Density lb/cu.ft	Percent of Max density	Depth of Test
	SUBGRADE SOIL						
	Lined Spray Evaporation Basins						
	East Basin						
5-14	N.E. Corner	10.5	118.1	7.9	113.8	96.3	0-8
5-14	S.W. Corner	10.5	118.1	9.7	112.3	95.1	0-8
5-14	Center of W. Side (sloped area)	10.5	118.1	8.1	109.0	92.3*	0-8
5-14	" Retest	10.5	118.1	8.7	110.6	93.6A**	0-8
5-145	N.Center of E. Side (sloped area)	10.5	118.1	11.9	112.6	95.3A	0-8
	West Basin						
5-15	N.E. Corner	10.5	118.1	9.2	113.6	96.2	0-8
5-15	S.W. Corner	10.5	118.1	7.3	114.2	96.7	0-8
5-15	S. Center of E. Side (sloped area)	10.5	118.1	9.1	111.7	94.6A	0-8
5-15	Center of W. Side (sloped area)	10.5	118.1	11.1	112.8	95.5A	0-8

NOTES:
1. Minimum compaction requirement 95% of a Modified Proctor Value 04 98% of a Standard Proctor.
A-92% Standard Proctor required, as per engineer.
2. Depth of test referenced in inches to top of subgrade.
*Test results fail to meet minimum requirement.**Retest results meet minimum requirement

ASTATULA LANDFILL
LAKE COUNTY, FLORIDA

RESULTS OF FIELD COMPACTION TESTS

JAMMAL & ASSOCIATES, INC. Consulting Engineers

Tested by:	PH	Date		Project No.	761-00273
Checked by:	DJD	Date	6/90	Sheet No.	3

Date 1990	Location of Test SUBGRADE SOIL	OMC %	Max Den lb/cu ft	Field Moisture %	Field Density lb/cu ft	Percent of Max density	Depth of Test
	Ash Residue Monofill Basin						
5-17	At lines F-2.5 (sloped area)	10.5	118.1	5.8	104.5	88.5A*	0-1
5-19	" Retest	10.5	118.1	10.2	103.8	87.9*	0-1
5-22	" Retest	13.7	107.5	13.3	105.6	98.2A**	0-1
5-17	At lines F-5	10.5	118.1	11.6	114.2	96.7	0-1
5-17	At lines H-5	10.5	118.1	4.7	115.9	98.1	0-1
5-19	At lines L.5-5	10.5	118.1	9.4	116.3	98.5	0-1
5-21	At lines G-8	10.5	118.1	6.6	113.9	96.4	0-1
5-21	At lines K-7.5	10.5	118.1	8.2	113.5	96.1	0-1
5-22	At lines K.5-2.5 (sloped area)	7.8	112.9	9.1	110.8	98.2A	0-1
5-22	At lines 0.25-6 (sloped area)	7.8	112.9	14.6	105.9	93.8A	0-1
5-22	At lines 0.25-10 (sloped area)	14.9	105.3	11.7	97.1	92.2A	0-1
5-22	At lines E-7	7.8	112.9	8.9	111.1	99.1A	0-1
5-22	At lines I-10	7.8	112.9	10.1	108.4	96.0	0-1
5-22	At lines M-10	10.5	118.1	8.4	112.5	95.3	0-1
5-22	At lines E.5 - 9 (ramp)	10.5	118.1	9.4	116.3	98.5	0-1
5-22	At lines E.5 - 10.5 (ramp)	10.5	118.1	6.8	114.1	96.6	0-1
5-23	At lines D.5 - 5 (sloped area)	7.8	112.9	9.2	107.3	95.1A	0-1
5-23	At lines I-1.8 (sloped area)	7.8	112.9	8.9	113.6	100+A	0-1
5-24	At lines F-9 (sloped area)	7.8	112.9	8.8	106.7	94.5A	0-1
5-24	At lines F-10.5 (sloped area)	13.7	107.5	9.1	98.2	91.4*	0-1

NOTES:
1. Minimum compaction requirement 95% of a Modified Proctor Value or 98% of a Standard Proctor value A-92% required, as per engineer.
2. Depth of test referenced in feet to top of subgrade.

*Test results fail to meet minimum requirement.**Retest results meet minimum requirement.

ASTATULA LANDFILL
LAKE COUNTY, FLORIDA

RESULTS OF FIELD COMPACTION TESTS

JAMMAL & ASSOCIATES, INC. Consulting Engineers

Tested by:	PH	Date:		Project No	761-00273
Checked by:	DJD	Date	6/90	Sheet No.	4

Date 1990	Location of Test CLAY LINER	OMC %	Max Den lb/cu ft	Field Moisture %	Field Density lb/cu ft	Percent of Max density	Depth of Test
	Lined Spray Evaporation Basins						
	East Basin						
	1st Lift						
5-14	85' S. & 16' E. of N.W. Corner	23.4	104.0	23.4	96.5	92.8*	0-4
5-15	" Retest	26.7	97.0	26.7	98.2	97.4**	0-4
5-15	61' N. & 32' E. of S.W. Corner	21.3	105.3	21.3	103.3	98.1	0-4
5-15	64' N. & 17' W of S.E. Corner	22.4	103.1	22.4	100.7	97.7	0-4
	2nd Lift						
5-17	44' N. & 37' E. of S.W. Corner	15.6	116.7	15.6*	113.8	97.5	0-4
5-18	" Retest			17.4*			
5-19	" Retest			20.1*			
5-22	" Retest	22.5	107.0	22.5**	106.2	99.2	0-4
5-17	53' S. & 29' E. of N.W. Corner	18.1	114.2	18.2*	111.6	97.7	0-4
5-18	" Retest			18.0*			
5-19	" Retest	22.0	109.7	22.0**	108.5	98.9	0-4
5-22	43' S. & 50' E. of N.W. Corner Retest B	21.6	106.0	21.6	103.4	97.5	0-4
	3rd Lift						
5-25	36' N. & 41' E. of S.W. Corner	26.0	103.3	26.0	100.0	96.8	14" total
5-25	62' S. & 53' E. of N.W. Corner	22.0	106.9	22.0	104.7	97.9	12-1/2" total

NOTES:
1. Minimum compaction requirement 97% of a 1 Point Modified Field Proctor
2. Depth of test referenced in inches to top of clay lift.

*Test results fail to meet minimum requirement.**Retest results meet minimum requirement.

ASTATULA LANDFILL
LAKE COUNTY, FLORIDA

RESULTS OF FIELD COMPACTION TESTS

JAMMAL & ASSOCIATES, INC. Consulting Engineers

Tested by: PH	Date:	Project No. 761-00273
Checked by: DJD	Date 6/90	Sheet No. 5

Date 1990	Location of Test	OMC %	Max Den lb/cu ft	Field Moisture %	Field Density lb/cu ft	Percent of Max density	Depth of Test
	CLAY LINER						
	Lined Spray Evaporation Basins						
	West Basin						
	1st Lift						
5-16	55' E. & 62' N. of S.W. Corner	21.0	107.6	21.0	104.6	97.2	0-4
5-16	26'W. & 50' S. of N.E. Corner	17.8	113.6	17.8	111.5	98.1	0-4
5-16	36' E. & 89' S. of N.W. Corner	19.6	112.1	19.6	109.6	97.8	0-4
	2nd Lift						
5-18	47' W. & 43' S. of N.E. Corner	20.8	110.2	20.8	106.6	96.8	0-4
5-18	67' W. & 24' W. of S.E. Corner	22.3	106.9	22.3	102.9	96.3*	0-4
5-18	" Retest	22.3	106.9	22.3	106.4	99.5**	0-4
	3rd Lift						
5-21	39' N. & 47' E. of S.W. Corner	25.5	100.6	25.5	98.7	98.1	13" total
5-21	37' S. & 43' E. of N.W. Corner	24.7	106.0	24.7	102.6	96.8	12" total

NOTES:
1. Minimum compaction requirement 97% of a 1 Point Modified Field Proctor
2. Depth of test referenced in inches to top of clay lift.

*Test results fail to meet minimum requirement.**Retest results meet minimum requirement.

ASTATULA LANDFILL
LAKE COUNTY, FLORIDA

RESULTS OF FIELD COMPACTION TESTS

JAMMAL & ASSOCIATES, INC. Consulting Engineers

Tested by:	PH	Date:		Project No	761-00273
Checked by:	DJD	Date:	6/90	Sheet No.	6

Date 1990	Location of Test	OMC %	Max Den lb/cu ft	Field Moisture %	Field Density lb/cu ft	Percent of Max density	Depth of Test
	CLAY LINER						
	Ash Residue Monofill Basin						
	1st Lift						
5-25	At lines M.5 - 2.5 (sloped area)	25.1	103.8	25.1	99.9	96.2*C	0-4
5-26	At lines H-3.5	22.6	108.8	22.6	106.6	98.0	0-4
5-26	At lines K.5 - 3.5	21.3	105.9	21.3	105.7	99.8	0-4
5-26	At lines F-2.5	24.9	100.1	24.9	99.6	99.5	0-4
5-26	At lines H-2.5	24.1	104.1	24.1	102.4	98.3	0-4
5-26	At lines K.5 - 2.5	22.1	104.9	22.1	103.9	99.0	0-4
5-26	At lines L.5 - 5	21.2	109.9	21.2	106.8	97.2	0-4

NOTES:
1. Minimum compaction requirement 97% of a 1 Point Modified Field Proctor
2. Depth of test referenced in inches to top of clay lift.

*Test results fail to meet minimum requirement.
C-Result accepted by engineer.

ASTATULA LANDFILL
LAKE COUNTY, FLORIDA

RESULTS OF FIELD COMPACTION TESTS

JAMMAL & ASSOCIATES, INC. Consulting Engineers

Tested by:	PH	Date:		Project No.	761-00273
Checked by:	DJD	Date:	6/90	Sheet No.	7

BIOGRAPHICAL SKETCH

Sadik Jaffer Al-musawe is a Kurd and was born in Baghdad, Iraq, where he completed his primary and secondary education. Then he moved to London, England. In 1975 he entered the civil engineering department at the Polytechnic of Central London and the Polytechnic of the South Bank. In 1980 he graduated with an equivalent to a Bachelor of Science degree in Civil Engineering. In 1981 he earned a Higher National Certificate (H.N.C.3) in highway engineering from Lincoln College of Technology in England. In 1982 he entered the graduate college at the University of Wisconsin in Madison, Wisconsin, and was awarded a Master of Science degree in mining and rock mechanics engineering (majoring in foundation engineering) in 1984. From 1986 to 1988 he entered the graduate school at the University of Tennessee and earned a number of graduate credits in civil engineering and engineering mechanics and science. In September 1988 he entered the graduate college (Geotechnical Engineering Division) at Georgia Institute of Technology at the doctoral level. In July 1989 he completed all the course work requirements. In August 1989 he transferred to the Department of Civil Engineering at the University of Florida

to complete his Ph.D. degree, majoring in geotechnical engineering with a minor in geology. This is due to the sudden reduction in the number of professors in the geotechnical department as well as the inavailability of opportunities for research involving soil mechanics. From 1974 to 1979 he worked in the soil and materials laboratory at the Polytechnic of Central London. From 1979 to 1981 he worked as a soil engineer with Humberside County Council in England. From 1981 to 1982 he worked as a geotechnical engineer with Law Engineering Testing Company in Saudi Arabia. Between 1983 and 1984 he was teaching assistant/instructor/foundation project assistant at the University of Wisconsin in Madison and Stoughton. From 1985 to 1986 he worked as a geotechnical engineer with Law Engineering Testing Company in Houston, Texas. From 1986 to 1988 he was the president of his own geotechnical and materials consultancy firm in Chattanooga, Tennessee. From 1988 to 1989 he worked as a private consultant to a number of firms in Chattanooga, Tennessee, and Atlanta, Georgia. From 1989 to 1990 he served as a teaching assistant in the Department of Civil Engineering at the University of Florida. He is a registered professional civil engineer and is a member of the American Society of Civil Engineers, Mining Engineers, and Petroleum Engineers.

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

Paul Y. Thompson, Chairman
Professor of Civil Engineering

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

David Bloomquist, Cochairman
Assistant Professor of Civil Engineering

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Anthony F. Randazzo
Professor of Geology

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

Wally H. Zimpfer
Associate Professor of Civil Engineering

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

Fazil T. Najafi
Assistant Professor of Civil Engineering

This dissertation was submitted to the Graduate Faculty of the College of Engineering and to the Graduate School and was accepted as partial fulfillment of the requirements for the degree of Doctor of Philosophy.

December 1990

for Winfred M. Phillips
Dean, College of Engineering

Madelyn M. Lockhart
Dean, Graduate School